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Strength of Joints that Combine Bolts and Welds

by

Thomas James Manuel ©

A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment of the requirements for the degree of Master of Science

in

Structural Engineering

Department Of Civil Engineering

Edmonton, Alberta

Spring, 1996

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The undersigned certify that they have read, and recommended to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "Strength of Joints that Combine Bolts and Welds," submitted by Thomas James Manuel in partial fulfillment of the requirements for the degree of Master of Science in Structural Engineering.

G.L. Kulak

G.L. Kulak, Supervisor

G. Grondin

G.Y. Grondin

Gary Faulkner

M.G. Faulkner

Jan. 25, 1996

ABSTRACT

In steel construction, it is occasionally necessary to combine both bolts and welds in a single joint. Provisions for the design of these combination joints can found in current standards, however, some are irrational because they proportion the mechanical fasteners to carry a percentage of the dead or live load and don't consider their ductility or positioning. In some cases, these design standards may prove to be unconservative. A better understanding of combination joints is required so that the provisions in current standards can be improved.

An experimental study using full-scale tension lap splices combining high-strength bolts and fillet welds was performed. Tests of the individual fastening elements were also done. The experimentation showed that the orientation of the welds and the bearing condition of the bolts are two key factors that must be considered when determining the extent of load sharing in combination joints. In this presentation, the test results of this study and the corresponding design recommendations will be discussed.

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LIST OF ABBREVIATIONS

AISC : American Institute of Steel Construction

ASD : Allowable Stress Design

ASTM : American Society for Testing and Materials

BSI : British Standards Institution

CSA : Canadian Standards Association

CEN : European Committee for Standarisations (Comité Européen de Normalisation)

ISO : International Standards Association

LRFD : Load and Resistance Factor Design

LVDT : Linear Variable Differential Transformer

MTS : Material Testing System

Sia : Swiss Society of Engineers and Architects (Schweizerischer Ingenieur - und Architektenverein)

LIST OF SYMBOLS

d = fillet weld leg size, mm

k_s = slip coefficient

l = length of fillet weld, mm

m = number of faying surfaces

n = number of bolts

P_{slip} = the slip resistance of a bolted joint, kN

$P_{U\theta}$ = ultimate strength of a fillet weld loaded in shear at any angle of loading, kN

P_θ = load acting on a fillet weld at any angle of loading, kN

R = shear resistance of bolt or fillet weld as a function of bolt deformation, kN

R_{bolts} = connection resistance contributed from bolt shear, kN

$R_{friction}$ = connection resistance contributed from plate friction, kN

R_{long} = connection resistance contributed from longitudinal weld shear, kN

R_{trans} = connection resistance contributed from transverse weld shear, kN

R_{ult} = ultimate shear resistance of a bolt, kN, or of a weld/length/leg size, kN/mm/mm

$R_{ult\ joint}$ = shear resistance of entire connection, as a function of average deformation, kN

T_i = initial clamping force of one bolt, kN

y = weld deformation, mm

x = distance from the centerline of the joint, mm

Δ = shear deformation of a bolt or a fillet weld, mm

Δ_{max} = ultimate deformation of a bolt or a fillet weld, mm

Δ_{Pu} = deformation at ultimate load $P_{U\theta}$ of a fillet at any angle of loading

λ = empirical regression coefficient

μ = empirical regression coefficient, mm^{-1}

θ = angle of weld with respect to direction of load, degrees

ρ = non-dimensional ratio Δ/Δ_{Pu}

1. INTRODUCTION

1.1 General

It is sometimes necessary to use combination joints in steel construction. A combination joint is a connection that utilizes more than one type of mechanical fastening element. For example, high-strength bolts can be used in combination with welds, rivets can be used in combination with welds, or rivets can be used in combination with high-strength bolts. In the literature, combination joints are also referred to as load-sharing joints, mixed joints, stress-sharing joints, and hybrid joints. Combination joints can be placed into two different categories: those that have differing mechanical fastening elements in the same shear plane, and those that have differing mechanical fastening devices in distinctly different shear planes. The analysis and research discussed here will only focus on the former case.

The need for a combination joint in steel construction can arise for a number of different reasons. For example, load requirements on an existing bolted joint may change, necessitating a renovation of that joint. As a result of bolt clearance requirements or limitations due to the existing connection plate size, there may not be sufficient space to add more bolts. Also, drilling a new bolt hole in the field may not be possible because of space restrictions. For this situation, adding welds to the connection is likely the only practical option available to give the connection the necessary increase in load resistance. The need for a combination joint may also arise in new construction. For assemblage of steel structures, the use of bolted connections rather than welded connections allows much higher tolerances in construction. It may be desirable to erect a steel structure using bolts proportioned in number to carry all the dead load of the steel frame. Upon erection of the frame, welds would be placed at all the connections to carry all the additional dead and live load. Another situation often requiring a combination joint is the renovation of a riveted connection. This may involve either the addition of high-strength bolts or actual replacement of rivets with high-strength bolts. This is common in old riveted structures in which additional connection strength is needed as a result of such factors as fatigue or new load requirements.

1.2 Statement of Problem

The strength of any connection is a function of the strength and deformation characteristics of each of the individual mechanical fastening elements present in the connection. Thus, these characteristics must be taken into account in order to accurately evaluate the strength of a combination joint. The assumption that the ultimate strength of a connection is the sum of the ultimate strengths of each individual mechanical fastening element present in the connection is clearly an unconservative approach to the design of the connection. In contrast, only accounting for the ultimate strength of one of the individual types of mechanical fastening elements present in a combination joint is apparently an extremely conservative approach. The answer to the question as to the real strength of the combination joint lies somewhere between these two approaches. Each of the different types of fastening elements has a different ductility; thus, each reaches its ultimate strength at a different overall connection deformation. In order to determine the ultimate strength of a combination joint, the load versus deformation characteristics of each mechanical fastening element must first be analyzed individually. Then it can be determined how these individual elements interact with each other and how much resistance each contributes to the connection.

A review of the literature indicates that there has not been much testing in the area of combination joints. The tests that have been done involve very small connections and these are limited to just one or two bolts. There are many conditions contained within a combination joint that can be varied. Such variations include the condition of the bolts at the start of loading (i.e., in bearing or not in bearing), the sequence in which welds and bolts are installed, the orientation of the welds (i.e., longitudinal or transverse), the differing load sharing contributions of snug-tight bolts as compared to pre-loaded bolts, and so on. Previous research involving combination joints has barely scratched the surface of the topic when considering all the various configurations in combination joints that can occur. More physical data involving many different configurations of combination joints are necessary in order to improve the understanding of their behaviour.

1.3 Objectives

A method of accurately predicting the load versus deformation response of combination joints is needed. Once this is available it should be possible to develop a safe procedure for the design of combination joints. As previously mentioned, however, combination joints can take many different forms. Developing an all-encompassing procedure that can be used to provide design rules for any combination joint will involve extensive research and analysis. Because of the limited scope of this research project , the method developed will necessarily be limited to kinds of combination joints with certain configurations. Using the results of a full-scale combination joint testing program, it is hoped that the validity of the method for predicting load versus deformation of the connection and the validity of the design procedure developed herein can be assessed. Recommendations made on types of combination joints not covered in this testing program can then be made on a rational basis; however, they will not be completely validated by physical testing data.

Different types of configurations will be investigated and tested as a part of this research. Differences in the connections such as initial bearing of the bolts and construction procedure will be considered. Although not every contingency can be investigated as a part of this study, the research presented here will perhaps generate more interest in this subject and prompt more research in the area of combination joints.

2. LITERATURE REVIEW

2.1 General

There has not been much research recorded that deals explicitly with the study of combination joints. The reports found that do deal specifically with combination joints will be discussed in detail. In addition to these sources, there are a number of papers dealing with the behaviour of the individual fastening elements found in combination joints. Also relevant to the study of combination joints is a review of various design standards from different countries, which give an explanation of how these countries deal with the design and fabrication of combination joints. A number of design standards are reviewed and are discussed in this chapter.

2.2 Behaviour of Bolts in Shear

In order to be able to predict the behaviour of combination joints, one must first be able to predict the behaviour of the individual fastening devices that make up the combination joint. Many different methods for predicting the load versus deformation response of bolts can be found in the literature. Crawford and Kulak [1971] performed a number of tests to determine the properties of bolts loaded in shear. Their tests used double lap splice connections and included tests using a compression jig as well as tests using a tension jig. The Crawford and Kulak tests used plates of ASTM grade A36 steel (nominal yield strength 248 MPa). The thicknesses of the lap plates and main plates used were 1/2 in. (13 mm) and 3/4 in. (19 mm), respectively. For a 3/4 in. diameter bolt of ASTM grade A325, these researchers reported that the average maximum shear resistance was 330 kN and the deformation at ultimate was 8.9 mm when tested in shear using a compression jig.

For the purpose of predicting the load versus deformation response of bolts, Crawford and Kulak [1971] used an analytical expression originally suggested by Fisher [1965]. The analytical model proposed by Fisher is as follows:

$$R = R_{ult} \times \left(1 - e^{-\mu\Delta}\right)^{\lambda} \qquad [2.1]$$

where: R = shear resistance of the bolt at any given deformation Δ

R_{ult} = ultimate shear resistance of the bolt

Δ = deformation

μ = empirical regression coefficient

λ = empirical regression coefficient

Using the Fisher [1965] model, Crawford and Kulak [1971] determined the coefficients μ and λ for a representative bolt using regression analyses. Based on the preceding bolt properties, Crawford and Kulak develop the following equation for one representative ASTM grade A325 bolt of 3/4 in. diameter (using units of kN and mm):

$$R = 330 \times \left(1 - e^{-0.394 \times \Delta}\right)^{0.55} \quad \text{for } \Delta \leq 8.9 \text{ mm} \qquad [2.2]$$

The *Guide to Design Criteria for Bolted and Riveted Joints* [Kulak et al., 1987] cites data from research done by Wallaert and Fisher [1965]. This research did not include testing of 3/4 in. diameter bolts; however, it was reported that the average shear strength and ultimate deformation for a 7/8 in. diameter A325 bolt is 80.1 ksi and 0.19 in., respectively. (The bolts were tested in double shear, and the average shear strength was found by dividing the maximum load by two times the shear area of the bolt.) The tension jig used by Wallaert and Fisher had two 1-in. lap plates and two 1-in. main plates that were of ASTM grade A440 steel (nominal yield strength 278 MPa). This is the only data given in the *Guide*, but a comparison with the 3/4 in. diameter A325 bolts can be made by computing the corresponding strength of 3/4 in. diameter bolt based on the shear stress obtained for the 7/8 in. diameter bolts. This comparison may not be completely valid, however, because different bolt diameters may exhibit slightly different shear strengths and deformations. However, using this comparison, the average maximum shear resistance for a 3/4 in. diameter bolt of ASTM grade A325 tested in shear using a compression jig is 314 kN and the deformation at ultimate is 4.9 mm.

The paper by Wallaert and Fisher [1965] does not give the regression coefficients for the analytical equation proposed by Fisher [1965], and therefore the overall bolt behaviour cannot be compared directly to that of Crawford and Kulak. The only data presented by them that are relevant to these studies are the ultimate loads and the deformations.

The analytical expression proposed by Fisher [1965] for the prediction of the load versus deformation behaviour of bolts in shear is the one generally accepted for use by researchers working with bolted joints. Moreover, not only is it used for predicting the behaviour of bolts, it is used for other types of mechanical fasteners. The model produces a load versus deformation equation that is easy to use and which emulates the actual behaviour of the load versus deformation response of a bolt in shear with good accuracy. Predicting the load versus deformation behaviour of a typical bolt, however, involves many factors. The deformation observed in a bolted connection subjected to load is the sum of the deformations resulting from shearing of the bolt, bending of the bolt, bearing of the bolt, and bearing on the plates. As a result, the bolt diameter, the plate strength, and the plate thicknesses are all relevant parameters when testing bolts in shear. Knowing the actual load versus deformation response of the bolts used in the full-scale test specimens is important for any testing program. Since there were no bolt test data available from tests that had exactly the same parameters as the full-scale specimens of this testing program, bolt tension tests and bolt shear tests were performed using bolts from the same lot as those used in the full-scale testing program. The two sets of bolt data presented by Crawford and Kulak [1971] and Wallaert and Fisher [1965] give a good baseline for comparison. In following chapters, test data from these two sources are compared to the data from the tests done as a part of the experimental testing program reported herein.

2.3 Behaviour of Fillet Welds in Shear

In the study of combination joints, it is necessary to know the individual behaviour of the fillet welds in the joint, just as it is necessary to know the individual behaviour of the bolts in the joint. As was the case with the analytical bolt equations previously discussed, there are a more than one set of load versus deformation equations for fillet welds that can be found in the literature.

Holtz and Kulak [1970] used Equation [2.1] proposed by Fisher [1965] to represent the load versus deformation behaviour of welds in combination joints. In turn, the regression coefficients and physical properties of the welds that they use were taken from research done by Butler et al. [1972]. Because it is known that weld strength and ductility are dependent on the angle of orientation of the weld with respect to the direction of loading, this angle must be incorporated into the equation for predicting the load versus deformation response of an individual weld. The empirical equations formulated by Butler et al. [1972], using S.I. units, are as follows:

$$R_{ult} = (27.6 \times 10^{-3}) \times \frac{10+\theta}{0.92 + 0.0603\theta} \times l \times d \qquad [2.3]$$

$$\Delta_{max} = 5.715 \times (\theta + 5)^{-0.47} \qquad [2.4]$$

$$\mu = 2.95 \times e^{0.0114 \times \theta} \qquad [2.5]$$

$$\lambda = 0.4 \times e^{0.0146 \times \theta} \qquad [2.6]$$

where: R_{ult} = ultimate shear resistance of the weld/length/weld leg size, kN/mm/mm

l = length of fillet weld, mm

d = leg size of fillet weld, mm

Δ_{max} = ultimate deformation of the weld, mm

θ = angle of weld with respect to direction of load, degrees

μ = empirical regression coefficient, mm^{-1}

λ = empirical regression coefficient

The preceding equations can be combined with Equation [2.1]. By substituting $\theta = 0°$ for the case of longitudinal welds, and $\theta = 90°$ for the case of transverse welds, respectively, the following equations result:

$$\frac{R}{l \times d} = 299.8 \times 10^{-3} \times (1 - e^{-2.95 \times \Delta})^{0.4} \quad [2.7]$$

$$\frac{R}{l \times d} = 434.5 \times 10^{-3} \times (1 - e^{-8.24 \times \Delta})^{1.49} \quad [2.8]$$

Miazga and Kennedy [1986] also performed tests on fillet welds oriented at various angles to the direction of the applied load. The welds that Miazga and Kennedy used in their tests were made using E48014 electrodes. Using the data from these tests, Lesik and Kennedy [1988] developed an equation to predict the load versus deformation response of fillet welds. This equation (using S.I. units) is as follows:

$$P_\theta = P_{U\theta} \times \begin{bmatrix} -13.29\rho + 457.32\rho^{1/2} - 3385.9\rho^{1/3} + 9054.29\rho^{1/4} \\ -9952.13\rho^{1/5} + 3840.71\rho^{1/6} \end{bmatrix} \quad [2.9]$$

where: P_θ = load acting on any fillet weld at any angle of loading, kN

$P_{U\theta}$ = ultimate strength of fillet weld loaded in shear at any angle of loading, kN

$= 0.291 \times d \times l \times [0.5 \times \sin^{1.5}\theta + 1.0]$ [2.10]

ρ = non-dimensional ratio Δ/Δ_{Pu}

Δ = deformation of a fillet weld at any angle of loading, mm

Δ_{Pu} = deformation at ultimate load $P_{U\theta}$ of a fillet at any angle of loading

$$= 0.209 \times d \times (\theta + 2)^{-0.32} \qquad [2.11]$$

θ = angle between the direction of the load and the axis of the weld, degrees

d = fillet weld leg size, mm

l = fillet weld length, mm

Equations [2.10] and [2.11] give the weld attributes used by Lesik and Kennedy [1988] in the equations of load versus deformation response from their analysis. For comparison with the weld attributes reported by Butler et al. [1972], and the weld attributes of the tests reported herein, the values of maximum shear strength have been normalized by dividing them by the nominal weld leg size and the nominal length of weld used in the tests. For welds oriented parallel to the direction of load, Lesik and Kennedy report an ultimate normalized shear strength of 0.437 kN/mm/mm and a deformation at ultimate of 0.30 mm. For welds oriented perpendicular to the direction of loading, they report an ultimate normalized shear strength of 0.292 kN/mm/mm and a deformation at ultimate of 1.0 mm.

Substituting the appropriate values into Equation [2.9], the following two equations predict the load versus deformation response for fillet welds.

$$\frac{P_{trans}}{l \times d} = 0.437 \times \begin{bmatrix} -13.29\left(\frac{\Delta}{0.295}\right) + 457.32\left(\frac{\Delta}{0.295}\right)^{1/2} - 3385.9\left(\frac{\Delta}{0.295}\right)^{1/3} \\ +9054.29\left(\frac{\Delta}{0.295}\right)^{1/4} - 9952.13\left(\frac{\Delta}{0.295}\right)^{1/5} + 3840.71\left(\frac{\Delta}{0.295}\right)^{1/6} \end{bmatrix} \qquad [2.12]$$

$$\frac{P_{long}}{1 \times d} = 0.292 \times \left[\begin{array}{c} -13.29\left(\frac{\Delta}{1.00}\right) + 457.32\left(\frac{\Delta}{1.00}\right)^{1/2} - 3385.9\left(\frac{\Delta}{1.00}\right)^{1/3} \\ +9054.29\left(\frac{\Delta}{1.00}\right)^{1/4} - 9952.13\left(\frac{\Delta}{1.00}\right)^{1/5} + 3840.71\left(\frac{\Delta}{1.00}\right)^{1/6} \end{array} \right] \qquad [2.13]$$

where: $P_{trans} = P_{\theta}$ for a value of θ equal to 90°

$P_{long} = P_{\theta}$ for a value of θ equal to 0°

The preceding methods all predict the load versus deformation response of fillet welds loaded in shear. A comparison of the two load versus deformation models is shown in Figure 2.1 and Figure 2.2. It should be noted that a direct comparison of these curves is not entirely valid because the welds in the two test series were made using different types of electrodes.

2.4 Behaviour of Combination Joints

2.4.1 Research by Holtz and Kulak

A series of tests involving combination joints was performed at Nova Scotia Technical College by Holtz and Kulak [1970]. In one series, connections that had high-strength bolts and welds in the same shear plane were tested in direct tension. A second series of tests was done using moment-resistant beam-to-column connections that had the bolts and welds in different shear planes. The discussion herein will only deal with the tension tests.

The tension splices tested by Holtz and Kulak [1970] were relatively small. Each connection contained either one or two 3/4 in. diameter A325 bolts high-strength bolts in combination with 1/4 in. fillet welds made with an E60xx (now E410xx) electrode, and each connection was in the form of a double lap splice. The lap plates used in the tests were 1/2 in. thick, and the main plates were 3/4 in. thick. All the tests used plates of

ASTM grade A36 (nominal yield strength 248 MPa). Nine specimens were tested in all, consisting of three identical splices in each of the following three groups:

1. Four welds parallel to the direction of the load, combined with two high-strength bolts. No clearance between the bolts and holes (so-called fitted bolts).
2. Four welds parallel to the direction of the load, combined with two high-strength bolts. Clearance between bolts and holes provided at the standard 1/16 in. (1.6 mm).
3. Two welds perpendicular to the direction of the load, combined with one high-strength bolt. Clearance between bolt and hole provided at the standard 1/16 in. (1.6 mm).

Holtz and Kulak [1970] made predictions of the load versus deformation behaviour of their combination joints using equations presented by Fisher [1965]. The ultimate deformations and strengths they used for their bolts and welds were the same as those used by Crawford and Kulak [1971] and Butler et al. [1972], respectively. (The bolts used by Holtz and Kulak were from the same lot as those used by Crawford and Kulak.) Thus, in predicting the behaviour of the individual bolts and individual welds, Equations [2.2], [2.7], and [2.8] were used.

Using these load versus deformation characteristics, the investigators calculated the resistance contributed from each individual fastening element for each increment of deformation of the combination joint. In the analysis, the deformation of the bolt was adjusted for the connections that had bolt hole clearances. The deformation of the bolt was taken as the overall deformation of the connection minus the bolt hole clearance. This adjustment is made because the bolts will not contribute any bearing resistance until all the connection plates have slipped with respect to each other a distance equal to the full bolt hole clearance. By summing up all the calculated resistances of the individual fastening elements, a prediction of the ultimate resistance of the joint was produced.

This method of prediction did not produce satisfactory predictions for the first series of tests, those which used fitted bolts. This may have been a result of a poor estimate of the original clearance of the bolts. In this series of tests, it was assumed that there was no slip between the time that the load is applied, and the time at which the bolts are in bearing. This implies that the bolts are contributing resistance right from the start of the test, which may not be an accurate prediction. In the second and third series of tests, where the standard 1/16 in. clearance was provided, a much better prediction of the test results was obtained. The results of these tests are shown Table 2.1, where the results have been converted to S.I. units.

Holtz and Kulak [1970] concluded that only a small amount of load, if any, was carried by the bolts at working loads (about 1/3 ultimate). In addition, they concluded that the effects of friction in preloaded bolts are too unreliable to be used in the prediction of the load versus deformation response. They stated furthermore that, although welds that are oriented perpendicular to the direction of load are stronger than those that are oriented parallel to the direction of load, they do not work very well in combination with bolts because of their limited ductility. Holtz and Kulak also predicted that the number of bolts used in a combination joint should be limited because they will not deform sufficiently to fully develop an appreciable amount of their strength.

2.4.2 Research by Jarosch and Bowman

Jarosch and Bowman [1985] performed a series of physical tests on combination joints at Purdue University. Their tests were similar to the tests performed by Holtz and Kulak [1970]. All of their test connections were a double lap splice, they were tested in tension, and they contained 3/4 in. diameter ASTM A325 bolts and/or fillet welds having a

1/4 in. nominal leg size and deposited using an E60xx (now E410xx) electrode. Unlike the Holtz and Kulak [1970] tests, however, Jarosch and Bowman did not test any connections containing fitted bolts, that is, they did not test any connection with bolts having zero hole clearance. The plate steel used was ASTM grade A36 (nominal yield strength 248 MPa), and the thicknesses of the lap plates and main plates were 1 in. (25 mm) and 5/8 in. (16 mm), respectively. Twelve specimens were tested, and the program consisted of two identical tests in each of the following six groups:

1. Two high-strength bolts only.
2. Four fillet welds only, oriented parallel to the direction of the load.
3. Two fillet welds only, oriented perpendicular to the direction of the load.
4. Four fillet welds oriented parallel to the direction of the load in combination with two high-strength bolts.
5. Two fillet welds oriented perpendicular to the direction of the load in combination with two high-strength bolts.
6. Four fillet welds oriented parallel to the direction of the load and two fillet welds oriented perpendicular to the direction of the load in combination with two high-strength bolts.

Jarosch and Bowman [1985] compared their test results with predictions from the same ultimate strength model used by Holtz and Kulak [1970]. Furthermore, they did not do any of their own ancillary tests. Rather, they used the same values of ultimate strength, ultimate deformation, and the same regression coefficients as did Holtz and Kulak.

The Jarosch and Bowman [1985] tests showed different behavioural properties within the scope of the connections tested. Their tests combining high-strength bolts with welds oriented parallel to the direction of loading showed behaviour consistent with their predicted behaviour. Slippage of the plates was observed, followed by attainment of the peak load. Afterwards, the load dropped off and then the connection failed. The tests in the final two test groups did not perform in the way that was predicted, however. In these tests, the bolts did not appear to contribute any resistance to the connection until the welds failed. Results of their tests are shown in Table 2.2.

As had been recommended by Holtz and Kulak [1970], Jarosch and Bowman also recommended that welds oriented perpendicular to the direction of load should not be used in combination with high-strength bolts because of their limited ductility at ultimate load.

2.4.3 Comments on Research into Combination Joint Behaviour

Both sets of tests reported here, Holtz and Kulak [1970] and Jarosch and Bowman [1985], add to the base of knowledge regarding the behaviour of combination joints, and their results are useful for comparison with results of any similar experimental program. The report by Holtz and Kulak [1970] gives insight into methods of prediction of behaviour of combination joints and into which types of combination joints are feasible or practical. The Jarosch and Bowman report uses the same prediction model as Holtz and Kulak, and the value of their work is the report of their six types of tests. Jarosch and Bowman reach the same conclusions as Holtz and Kulak.

2.5 Current Design Specifications

2.5.1 Specifications Reviewed

In order to better understand the behaviour of combination joints, a number of design specifications from around the world are examined herein. The design standards that were examined as a part of this research included the Canadian standard [CSA, 1994], the two American standards [AISC 1989; AISC, 1993], an international draft standard [ISO, 1994], the British standard [BSI, 1985], the Swiss standard [Sia, 1979], and the

European Committee for Standardisation [CEN, 1992]. The following sections give a brief outline of how each code deals with the design of combination joints.

2.5.2 Combination Joints with Bearing-Type Connections

Almost all of the design standards that were reviewed are in agreement on how to design new connections combining high-strength bolts for bearing-type load transfer with welds. All of the standards, except for the Swiss [Sia, 1979], specifically state that no load sharing will be considered in these types of connections, meaning that one of the fastener types must be proportioned to carry all the load. The two American standards, allowable stress design (ASD) [AISC, 1989] and load and resistance factor design (LRFD) [AISC, 1993], go one step further and say that the welds must be proportioned to carry the entire force in the connection. Thus, for these types of connections, the American standards disallow counting on resistance from the bolts in any situation.

The Swiss standard [Sia, 1979], on the other hand, takes an entirely different approach to combination joints than the other standards. It allows the designer to assign as much or as little load-sharing in any combination joint as seen fit. The relevant Swiss standard code provision reads as follows:

> "Where combined use of different types of fastening is made in the same connection, the carrying capacity shall take into account the real mode of transmission of forces and in particular the differences in deformation. This is also valid for connections made with one type of fastening whose constructional realization leads to unequal deformations of its different parts."

Thus, the Swiss design standard leaves it up to the designer to determine the deformation of each individual fastening device present in the connection, and to evaluate how much resistance each fastening element will contribute to the connection.

2.5.3 Combination Joints with Preloaded High-Strength Bolts

All of these standards allow high-strength bolts in slip-critical connections to share load with welds to some extent. In the design of such connections, the Canadian S16.1 standard [CSA, 1994] is perhaps the most conservative of all the standards reviewed. High-strength bolts proportioned for slip critical connections may share specified load with welds; however, the load sharing will be on the basis of the proportional capacities of the bolts in the connection and 0.70 times the factored resistance of the welds. This factor imposed on the weld capacity is applied because bolt capacity is based on the slip criterion at the specified load level, and welds are usually proportioned for factored loads.

The International Standards Organization rules [ISO, 1994] also allow high-strength bolts proportioned for slip-critical connections to share load with welds at the specified load level. Unlike the Canadian standard [CSA, 1994], however, it does not restrict the proportional capacity of the factored weld resistance. It does, nevertheless, state specifically that this load sharing can only be considered if the final tightening of the high-strength bolts occurs after the welds are deposited.

There appears to be a discrepancy, among these design standards, about the order in which the high-strength bolts should be tightened and the welds should be placed. Although ASD [AISC, 1989] and LRFD [AISC, 1993] both allow load sharing of welds with preloaded high-strength bolts, in both cases their Commentary recommends that the final tightening of the preloaded bolts should be done before the welds are deposited. The British Standard [BSI, 1985], the International standard [ISO, 1994], and the European standard [CEN, 1992] all state that in order to consider load sharing in this type of connection the final tightening of the bolts must be done after the welding is completed. Provisions found in the Canadian S16.1 standard [CSA, 1994] and the Swiss standard [Sia, 1979] discuss this issue.

The two American standards [AISC, 1989; AISC, 1993] mention this issue in their Commentary. However, there is no requirement for such a procedure in the provisions of the code itself. Furthermore, the Commentary merely makes a recommendation to do the final tightening of the bolts before the welding is done. They do, nonetheless, provide reasoning as to why the bolts should be tightened first; their rationalization is that the welds may interfere with the high contact pressure of the bolts required in a slip-critical connection. In addition, these specifications state that the weld heat will not alter the mechanical properties of the bolts.

The remaining standards that address this issue do not allow load-sharing of plate friction from preloaded bolts with shearing of the welds unless the final tightening of the bolts is done before the welds are placed. It is not apparent what the rationale is behind this design procedure because there were no commentaries included in these design standards and no references were given revealing the source of their information.

2.5.4 Welded Alterations to Existing Bolted or Riveted Connections

According to the Canadian S16.1 standard [CSA, 1994], welds added to existing joints with bolts or rivets should be proportioned to carry all the load except the original dead load. The LRFD standard [AISC, 1993] states that welds added to existing joints with bolts or rivets tightened to the requirements for slip-critical connections should be proportioned to carry all the load except the original dead load. The existing dead load can be carried by the bolts or rivets already present. The ASD standard [AISC, 1989] is similar to LRFD standard in that it states the existing bolts or rivets can carry the "loads present at the time of alteration." This implies that both existing dead and live load may be carried by the bolts or rivets already present, provided they have a clamping force that is consistent with the requirements of a slip-critical connection. The latter two design standards are different from the Canadian standard in that they only allow this type of load-sharing if the welds are added to slip-critical connections. As already mentioned, the Canadian standard does not stipulate this condition.

Proportioning the welds and bolts in a combination joint to carry a percentage of the dead or live load does not seem to be a rational way of designing a connection. The percentage of live and dead load on a connection can change significantly from one situation to another. Thus, the connection should be designed for each specific case. Designing a connection by using a general rule that assigns a particular percentage of load resistance to each mechanical fastening device, based on dead and live load could result in an unconservative design.

No mention of welded alterations to bolted or riveted joints was found in the remaining standards that were reviewed. However, as stated earlier, they do stipulate that in order to consider load-sharing, tightening of bolts in slip-critical connections must be done after welding is completed. Thus, in the situation where welds are added to a slip-critical (or, friction-type) bolted connection, these standards can be interpreted as saying the welds must be proportioned to carry all the load. Furthermore, when welds are added to a bearing-type connection, the same requirements must be met as is the case with new work.

Table 2.1 Test Results from Research by Holtz and Kulak [1970]

Specimen No.	Ultimate Load kN		Ratio of Predicted to Test Load	Allowable Load (CISC, AISC) kN	Factor of Safety	Ultimate Deformation mm
	Test Load	Predicted Load				
BW-L-0-1	1170	1303	1.11	258	4.53	2.1
-2	996	1254	1.26	241	4.13	1.3
-3	952	1201	1.26	225	4.24	–
BW-L-1-1	1117	1130	1.01	367	3.04	2.0
-2	1090	1113	1.02	362	3.01	2.2
-3	1174	1148	0.98	374	3.14	2.3
BW-T-1-1	1317	1197	0.91	334	3.94	–
-2	1334	1286	0.96	355	3.75	–
-3	1446	1334	0.92	367	3.93	–

Table 2.2 Test Results from Research by Jarosch and Bowman [1985]

Specimen No.	Test Load kN	Computed Loads, kN			Ratio of Computed Load to Test Load
		Weld Load	Bolt Load	Total Load	
W0B2-1	678	0	658	658	0.971
W0B2-2	681	0	658	658	0.967
WLB0-1	790	758	0	758	0.961
WLB0-2	849	808	0	808	0.952
WTB0-1	632	593	0	593	0.939
WTB0-2	852	764	0	764	0.897
WLB2-1	1161	926	370	1295	1.116
WLB2-2	1094	903	370	1273	1.163
WTB2-1	778	809	0	809	1.039
WTB2-2	812	803	0	803	0.989
LTB2-1	1559	1685	0	1685	1.080
LTB2-2	1446	1646	0	1646	1.138

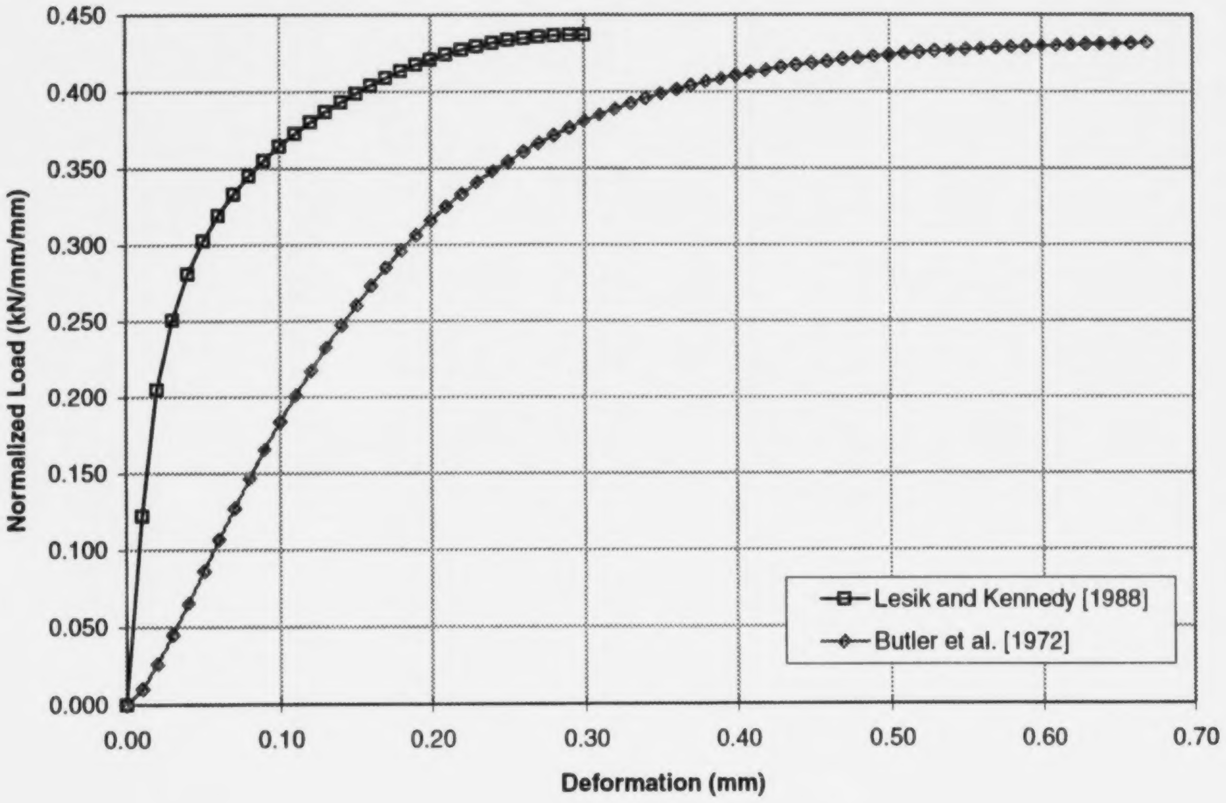

Figure 2.1 Normalized Load Versus Deformation : Fillet Weld ($\theta = 90^{o}$)

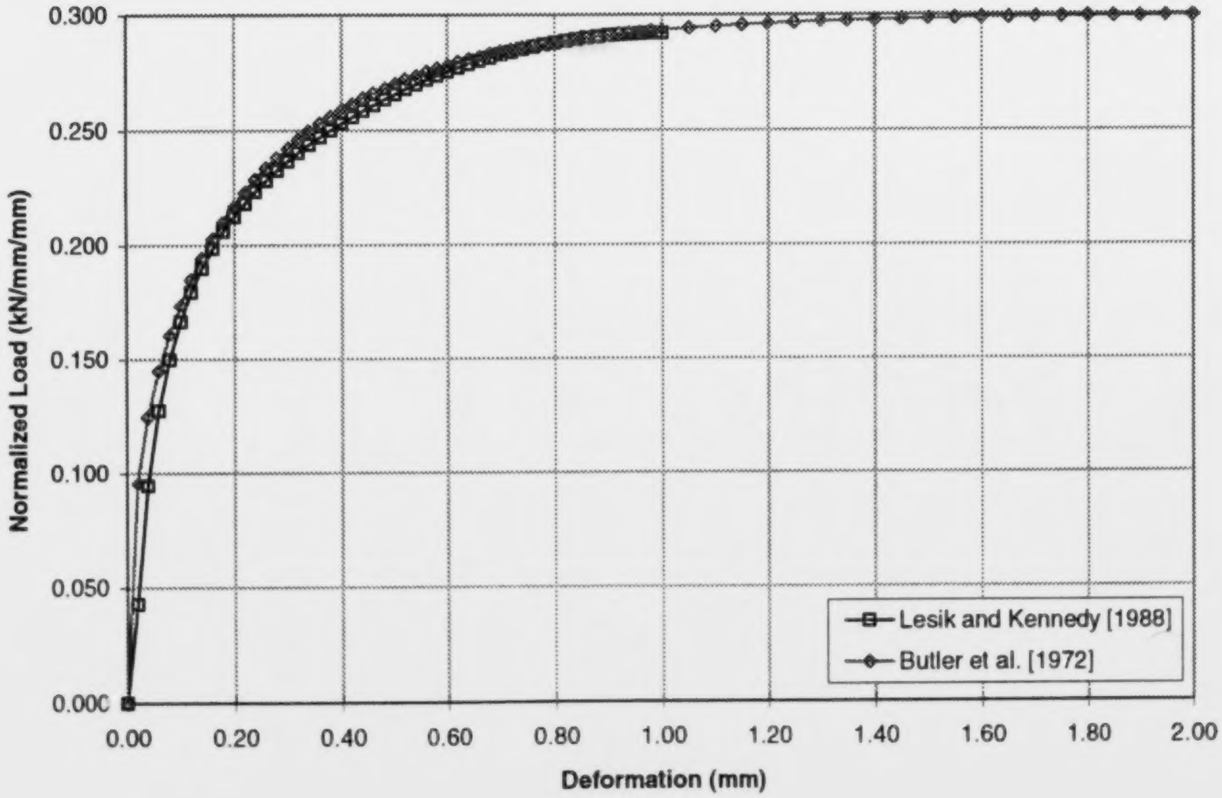

Figure 2.2 Normalized Load Versus Deformation : Fillet Weld ($\theta = 0^{o}$)

3. EXPERIMENTAL PROGRAM

3.1 General

In order to accurately predict the ultimate strength of combination joints, it is necessary to be able to predict the load versus deformation response of the joint. In turn, this requires that a considerable amount of physical testing must be done. In some of the previous research efforts, the size of the specimens and the scope of the physical testing program were limited and the results are thereby restricted in usefulness. For example, although the test results presented by Holtz and Kulak [1970] and by Jarosch and Bowman [1985] are of value, all of their test specimens were small in size and were restricted to one or two bolts. Combination joints can exist in various configurations and sizes, and thus many different tests entailing a variety of sizes and configurations are needed so that test results can be added to the data accumulated thus far.

In order to investigate full-scale combination joints and to add to the available test data, a testing program was performed as a part of this research project. The main purpose of the experimental program was to verify the predictions made for the load versus deformation response of combination joints of the specific configurations under study in this research. In addition, the ultimate strength and deformation of these particular joints can also be evaluated on the basis of the research done by others. The testing program consisted of two parts: the main testing program and an ancillary testing program. The main testing program was for the testing of full-scale tension lap splices that have both welds and bolts acting as mechanical fastening elements in the same shear plane. The ancillary testing program was for the testing of the individual fastening elements and to obtain the relevant properties of the steel used in the main testing program.

3.2 Parametric Variables

The following is an outline of parameters that must be taken into account for determining the load versus deformation response of combination joints. By varying these parameters, many different types of combination joints showing differing strengths and deformations can be investigated. The parameters investigated include:

1. The angle of the fillet weld with respect to the direction of the applied load.
2. The position of the bolts with respect to the bolt holes in the plates.
3. The method of installation of the bolts and the corresponding bolt pretension.
4. The order of installation of the welds and the bolts.

Because fillet welds exhibit varying resistances when loaded at different angles, the angle of the fillet weld with respect to the direction of loading is an important factor in determining the strength of a combination joint As reflected in the S16.1 Standard [CSA, 1994], the shear strength of a fillet weld oriented perpendicular to the line of force (a *transverse* weld), is approximately one and a half times that of a weld oriented parallel to the line of force (a *longitudinal* weld). In addition to exhibiting differing shear strengths, welds loaded at different angles also exhibit significantly different deformation characteristics. Both longitudinally loaded fillet welds and transversely loaded fillet welds were investigated in the experimental program discussed herein. Different welds having these two types of configurations (transverse and longitudinal) were tested by themselves, each in combination with bolts, and together in combination with bolts.

The position of the bolts with respect to the bolt holes in the plate is another important element in determining the strength and behaviour of combination joints. This condition is a key factor in determining how much load sharing exists between the bolts and the welds in the connection. In the worst case situation, the connected parts will have to slip an amount equal to two hole clearances before the bolts will be in contact with the connected material (see Figure 3.1). This case will be referred to as *negative bearing*. In the best case, the bolts will be in contact with the sides of the holes as soon as force is applied to the joint. In this condition, the bolts contribute bearing resistance to the applied force from the start of loading (see Figure 3.2). This case will be referred to as *positive bearing*. Both of these cases were included as a part of the testing program.

Another factor that influences the strength of both combination joints and joints that are only bolted, is the method of installation of the bolts. When all plies in a joint are in firm contact but the bolts have not been preloaded, it is expected that slip will not be prevented as load is applied to the joint. In this case, the bolts are in a condition defined as "snug-tight" [CSA, 1994]. This condition is achieved by a few impacts of an impact wrench, or by the full effort of a person using a spud wrench [CSA, 1994]. For so-called bearing-type connections, a snug-tight condition is all that is required. For slip-critical connections, however, a method of preloading the bolts is required. Preloading can be achieved by means of the turn-of-nut tightening method, or by use of some type of direct tension indicator. The turn-of-nut tightening method requires that all of the bolts in the joint be first brought to a snug-tight condition, followed by tightening each bolt additionally by the applicable relative rotation for that size and length of bolt [CSA, 1994]. Combination joints containing preloaded bolts and combination joints containing bolts tightened only to the snug-tight condition were each investigated as a part of the testing program. These are referred to as having a *preloaded* condition or a *snug-tight* condition, respectively. The method of preloading that was used in the testing program reported herein was the turn-of-nut method.

The order in which the welds and bolts are installed with respect to each other may also be an important consideration. In the construction of combination joints, the bolts may be installed before the welds, or vice versa. As mentioned in Chapter 2, this seems to be an area of difference among different design standards. Some of the standards examined specify that preloaded bolts should be tightened before the welds are deposited, while others indicate the opposite. Although this factor may be important in the study of combination joints, it is not part of the scope of this research and was therefore not investigated as a part of the testing program. This is a topic worthy of future investigation.

3.3 Ancillary Testing Program

3.3.1 General

A series of ancillary tests was performed in order to investigate the behaviour of the individual elements found in the full-scale combination joints. The tests done as a part of the ancillary testing program were performed in order to determine the shear strength and corresponding shear deformation of the bolts, the tensile strength of the bolts, the shear strength and deformation of longitudinally oriented fillet welds and of transversely oriented welds, and the tensile strength of the steel plate. A total of twenty-three tests were done in the ancillary testing program.

All of the steel used in the fabrication of the ancillary test specimens was from the same source as subsequently used in the full-scale test specimens, and it was grade 300W (CAN/CSA-G40.21-M92). The bolts used in the bolt shear and bolt tension tests were also from the same manufacturing lot as subsequently used in the full-scale test specimens. These bolts were of ASTM grade A325 and were 3/4 in. diameter. The bolt length was selected such that the bolt threads were not in any shear plane when the joint was loaded. The fillet welds used in the tests have a nominal weld leg size of six millimeters and were deposited using E48018-1 weld electrodes; as was the case with the full-scale test specimens. All of the specimens in the ancillary testing program were tested in a 1000 kN capacity materials testing system (MTS 1000), which contains an internal load cell that measures the applied load.

3.3.2 Bolt Tension Tests

In order to determine the tensile strength of the bolts being used in the full-scale test specimens, tensile tests were performed on individual bolts taken from the same lot. The bolts were tested using a compression jig (see Figure 3.3), and monitored using one linear variable differential transformer (LVDT), the testing machine stroke, and the internal load cell of the testing machine. Three single bolts were tested using the procedure recommended in ASTM standard A325-92a [ASTM, 1992].

3.3.3 Bolt Shear Tests

The factors reflecting the shear behaviour of bolts include the shearing of the bolts, the bending of the bolts, the bearing of the bolts, and the localized deformation of the plates in the connection. In order to accurately model the combination connection, it is essential that the ancillary bolt shear test be carried out using the same dimensions and bolt and plate material characteristics that will be used subsequently in the full-scale tests. The full-scale specimens of the testing program reported herein had characteristics unlike any specimens reported in the literature, so bolts from the same lot as those used in these specimens were individually tested in shear. The bolts were loaded in double shear using a tension jig installed in the testing machine by means of grips (see Figure 3.4). The load and deformation was recorded using two LVDT's and the internal load cell of the testing machine.

The procedure used in testing the specimens was as follows. Each tension jig was first assembled with the bolts tightened only hand-tight. The tension jig was then installed in the testing machine and all the pieces were aligned parallel with respect to each other and with the testing machine. A small tension load was then applied to the tension jig to ensure that the bolts were in bearing from the start of loading. While being subjected to this small load, the bolts were tightened to a snug-tight condition. The load was brought back down to zero, then the entire apparatus was again loaded; this time until failure. The load and the deformation were both recorded at small, regular intervals. When the peak load was reached, the LVDT's were removed to avoid damage.

3.3.4 Weld Shear Tests

3.3.4.1 General

Weld shear tests were performed in order to establish the shear strength and deformation response of welds made using the same weld electrode as that used subsequently in the full-scale testing program. Two different types of weld shear tests were performed: one test was designed to test transverse welds and the other was designed to test longitudinal welds. The test specimens were double lap splices (Figure 3.5 and Figure 3.6), and they were accommodated within the testing machine by means of hydraulic grips. All surfaces to be welded were first prepared by removing any mill scale with a hand-held grinder.

Each of the specimens was prepared and tested using the same procedure. First, the specimen was assembled by tightening the fixture bolts (Figure 3.5 and Figure 3.6) only to a hand-tight condition. Next, the specimen plates were aligned, and the fixture bolts were tightened to a snug-tight condition. The plates requiring welding were then clamped together and fillet welds with six millimeter nominal leg sizes were deposited on the test specimens at the appropriate locations. Plaster casts of the welds were taken, and the leg sizes were measured at length intervals of approximately 25 mm. The specimen was thereupon installed in the testing machine and all the pieces were aligned parallel with respect to each other and with the testing machine. The specimen was loaded in tension until failure; the load and the deformation were both recorded at small, regular intervals. The deformation for these tests was measured using two LVDT's, and the load was measured using the internal load cell of the testing machine. When the peak load was reached, the LVDT's were removed to avoid damage.

3.3.4.2 Longitudinal Welds

In the longitudinal weld shear testing series, each specimen had four welds that were nominally 60 mm in length. Although there were five weld coupons fabricated for this series, one specimen was ruined during test preparation and therefore only four could be tested. Each specimen had dimensions consistent with those shown in Figure 3.5.

3.3.4.3 Transverse Welds

Each transverse weld shear specimen had two welds that were nominally 75 mm long. In depositing the welds, start-up and run-off tabs were used to ensure a consistent weld throughout the 75 mm weld length. There were five transverse weld shear specimens tested and they had dimensions consistent with those shown in Figure 3.6.

3.3.5 Steel Tension Test

In order to determine the static yield stress and the ultimate stress of the steel plate used subsequently in the full-scale testing program, six tension coupons were tested. All six coupons were machined to dimensions consistent with ASTM Standard A370-92 [ASTM, 1992], and are shown in Figure 3.7. Corresponding to the thicknesses of the full-scale testing program, three of the coupons were nominally 19 mm thick, and three were 35 mm thick.

Each test was monitored with two strain gauges, an extensometer, the testing machine stroke, and the internal load cell within the testing machine. The tension coupons were tested in the MTS 1000, and were held by means of the hydraulic grips of the testing machine. The rate of strain used was approximately 5 $\mu\varepsilon$/sec for the elastic range and approximately 50 $\mu\varepsilon$/sec for the plastic range. The load and the deformation were recorded at small, regular intervals. When yielding started, the load was held constant for about five minutes in order to obtain the static yield stress.

3.4 Main Testing Program

3.4.1 Specimen Description

Twenty-four full-scale combination joints were tested in direct tension in a 6000 kN capacity materials testing system (MTS 6000). Each of these specimens combined four 3/4 in. diameter bolts of ASTM grade A325 with fillet welds of six millimeter nominal weld leg size. All of the specimens had dimensions consistent with those shown in Figure 3.8, and examples of typical test specimen set-ups are shown in Figure 3.9 and Figure 3.10. The exact configurations of the joints varied, as outlined in Section 3.2. All of the steel plate used in the test specimens was expected to meet the requirements of CAN/CSA-G40.21-M92 300W grade, and it all came from the same heat.

Three different types of weld arrangements were used in the 24 specimens. The first eight specimens tested bolts in combination with four longitudinal fillet welds, each 140 mm long. The second eight specimens had bolts tested in combination with two transverse fillet welds, each 260 mm long. The last group of tests consisted of eight specimens containing bolts and both longitudinal and transverse welds of the same lengths as in the first two groups.

Preparation of the test plates included the removal of mill scale with a hand-held grinder in the areas to be welded.

This testing program was conducted to test and predict the combined resistance of the bolts and the welds. Therefore, in designing the connections, the welds and bolts were proportioned to have a lower combined resistance than the factored resistance of the plates. This ensured that the fasteners in the joints failed before the plates. Each of the test specimen joints had two 19 mm thick lap plates connected to a 35 mm thick main plate by means of welds and high-strength bolts. There were also two fixture plate assemblies within each test specimen. These provided a means of attaching the test specimen to the testing machine. Each fixture plate assembly was fabricated by welding three plates together and then drilling a 90 mm diameter pin-hole and eight 27 mm diameter bolt holes. The pin-hole was used for attaching the entire specimen to a clevis on the testing machine; the bolts holes were used for attachment of the fixture plate assembly to the test connection. The same two assemblies were used for each of the 24 full-scale combination joint tests, and were designed such that they would not yield or fracture before any of the test connection plates.

3.4.2 Specimen Designation

A nomenclature was developed for the designation of each of the twenty-four main test specimens. The system is based on the weld arrangement present in the specimen and other features of the configuration. The system consists of three letters, followed by a dash, followed by a number. The system is as follows:

X X X – X

Specimen number of that series.
Weld arrangement : L denotes longitudinal welds only
T denotes transverse welds only
A denotes both transverse and longitudinal welds
Bolt installation technique : P denotes preloaded bolts
S denotes snug-tight bolts
Bearing condition of bolts : N denotes negative bearing
P denotes positive bearing

A complete listing of the specimens tested, along with their attributes, is given in Table 3.1.

3.4.3 Instrumentation

The load on the test specimens was measured using the internal load cell of the testing machine. Measurements of the relative displacement of the lap plates with respect to the main plates were taken at various points in order to estimate the deformation of the individual mechanical fastening devices present in the connections. For this purpose, each test utilized ten LVDT's placed at various locations on the specimen. Each LVDT was attached to a small steel tab tack-welded to the specimen. The location of the LVDT's on a specimen was dependent on the weld arrangement used for each particular test. The test specimens with only longitudinal welds had the LVDT's oriented as shown in Figure 3.11. Test specimens with only transverse welds and test specimens with both transverse and longitudinal welds had the LVDT's oriented as shown in Figure 3.12.

At selected locations on the test specimen, manual readings of deformation were taken using a depth gauge. These readings were taken when any of the individual mechanical fastening devices fractured before the entire connection failed. For example, if one of the welds in the connection fractured but the bolts still remained intact, then the experiment was temporarily stopped to take depth gauge readings. Occasionally, the LVDT's shifted slightly within their sheaths as a result of energy being released when an individual mechanical fastener fractured. In such cases, measurement with a depth gauge was carried out as a precautionary measure in order to be able to adjust the LVDT values later if necessary.

3.4.4 Testing Procedure

The following is an outline of the procedure used for testing the full-scale combination joint tests:

1. The plate assembly was placed into the testing machine. The bolts were only hand-tight at this time.

2. The specimen was subjected to a small tensile load in order to properly align the specimen plates with respect to each other and with the testing machine.
3. The bolts connecting the fixture plate assemblies to the test specimen plates were tightened to snug-tight condition, then rotated a one-third turn rotation using an impact wrench.
4. a) In tests with bolts in a negative bearing condition, a small compressive load o approximately 50 kN was applied to the test specimen.
    b) In tests with bolts in a positive bearing condition, a tensile load was applied to test specimen. This tensile load was increased until none of the bolts were able slide freely out of the bolt holes. Fabrication tolerances were quite small, and t did not require very much load relative to the ultimate load of the connection.
5. The bolts in the test connection were tightened to snug-tight condition with a hand wrench. For tests requiring preloaded test bolts, an additional one-half turn rotation was applied to the test bolts using an impact wrench.
6. Welds were deposited on the test specimen according to the weld arrangement required for that particular specimen. Plaster casts of the welds were then taken at length intervals of approximately 25 mm.
7. In order to be able to observe the yielding patterns of the steel, all steel in the test specimen was painted with whitewash.
8. Instrumentation was installed on the test specimen.
9. The specimen was loaded in tension and readings of load and deformation were taken at regular intervals. In order to avoid damaging the LVDT's, all but two of them were removed at the point of imminent failure of the entire connection. The test specimen was then loaded to failure.

Table 3.1 Main Testing Program Test Specimens

Specimen No.	Number of Bolts	Length of Transverse Welds, mm	Length of Longitudinal Welds, mm	Bearing Condition	Bolt Installation Procedure
NSL-1	4	—	560	Negative	Snug-Tight
NSL-2	4	—	560	Negative	Snug-Tight
NPL-1	4	—	560	Negative	Preloaded
NPL-2	4	—	560	Negative	Preloaded
PSL-1	4	—	560	Positive	Snug-Tight
PSL-2	4	—	560	Positive	Snug-Tight
PPL-1	4	—	560	Positive	Preloaded
PPL-2	4	—	560	Positive	Preloaded
NST-1	4	520	—	Negative	Snug-Tight
NST-2	4	520	—	Negative	Snug-Tight
NPT-1	4	520	—	Negative	Preloaded
NPT-1	4	520	—	Negative	Preloaded
PST-1	4	520	—	Positive	Snug-Tight
PST-2	4	520	—	Positive	Snug-Tight
PPT-1	4	520	—	Positive	Preloaded
PPT-2	4	520	—	Positive	Preloaded
NSA-1	4	520	560	Negative	Snug-Tight
NSA-2	4	520	560	Negative	Snug-Tight
NPA-1	4	520	560	Negative	Preloaded
NPA-2	4	520	560	Negative	Preloaded
PSA-1	4	520	560	Positive	Snug-Tight
PSA-2	4	520	560	Positive	Snug-Tight
PPA-1	4	520	560	Positive	Preloaded
PPA-2	4	520	560	Positive	Preloaded

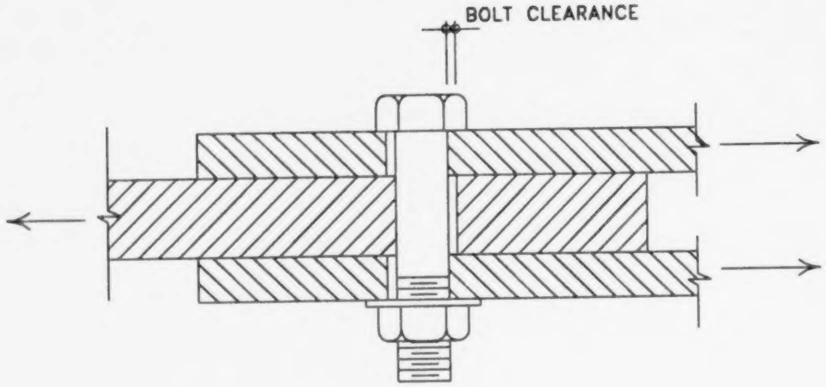

Figure 3.1 Bolt in Negative Bearing

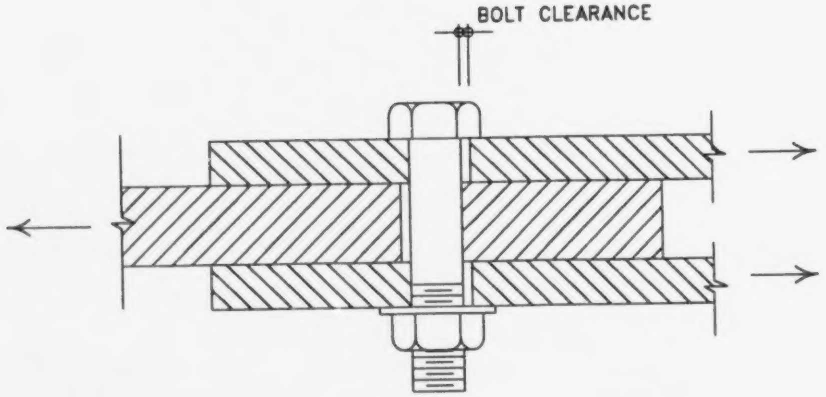

Figure 3.2 Bolt in Positive Bearing

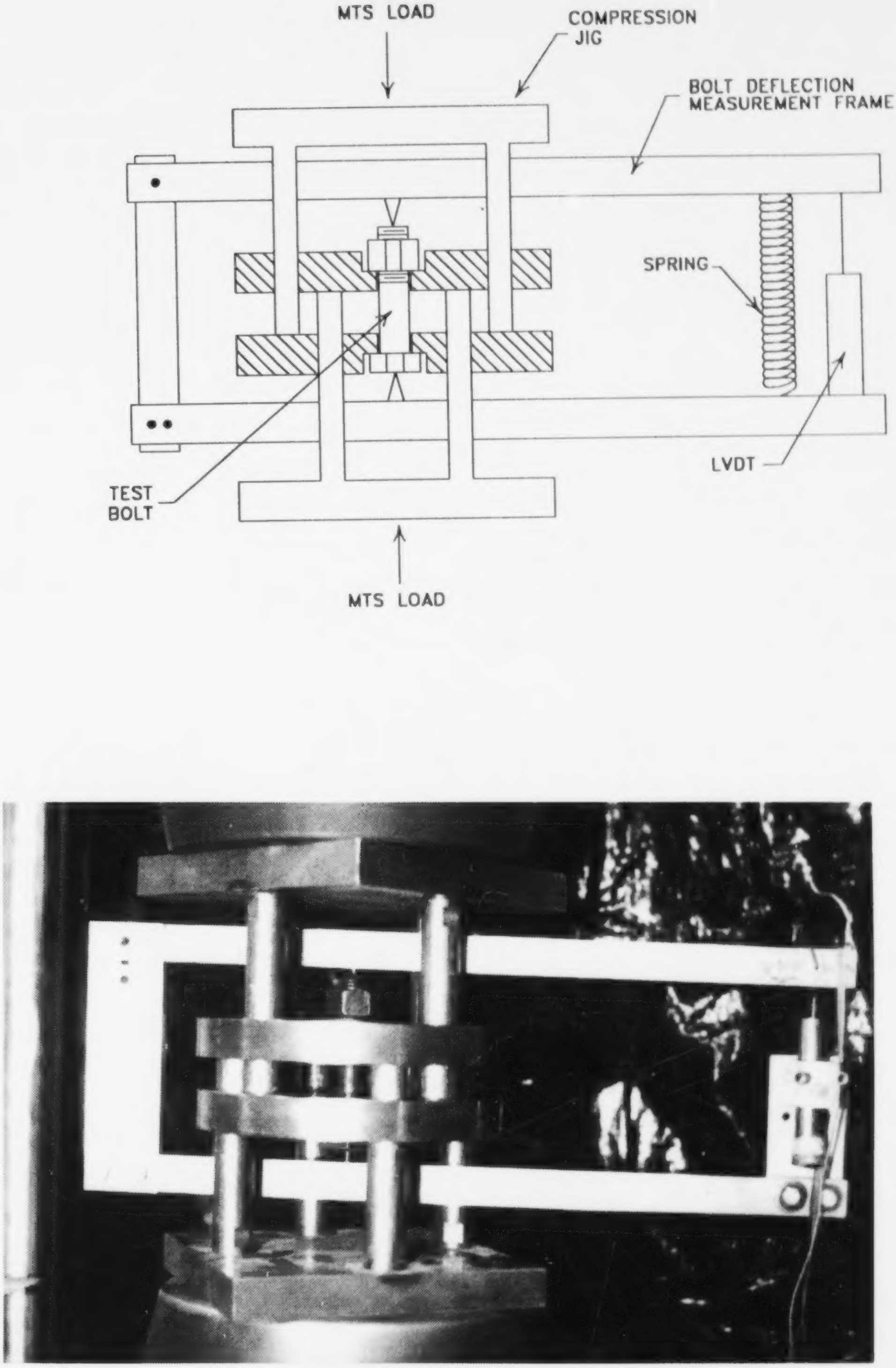

Figure 3.3 Bolt Tension Test Apparatus

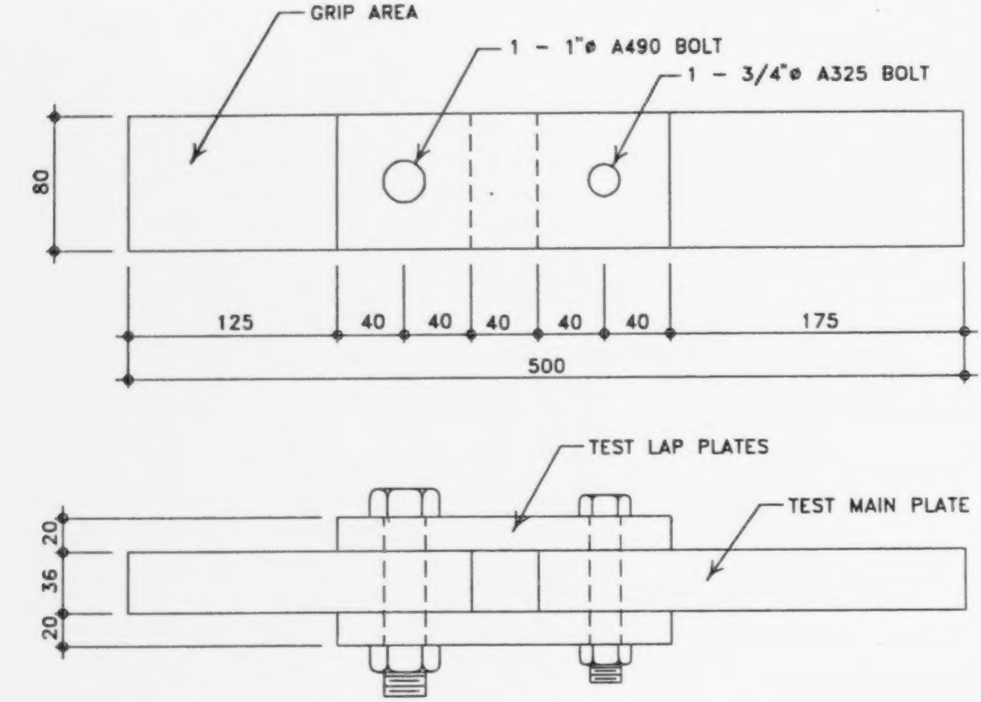

Figure 3.4 Bolt Shear Tension Jig

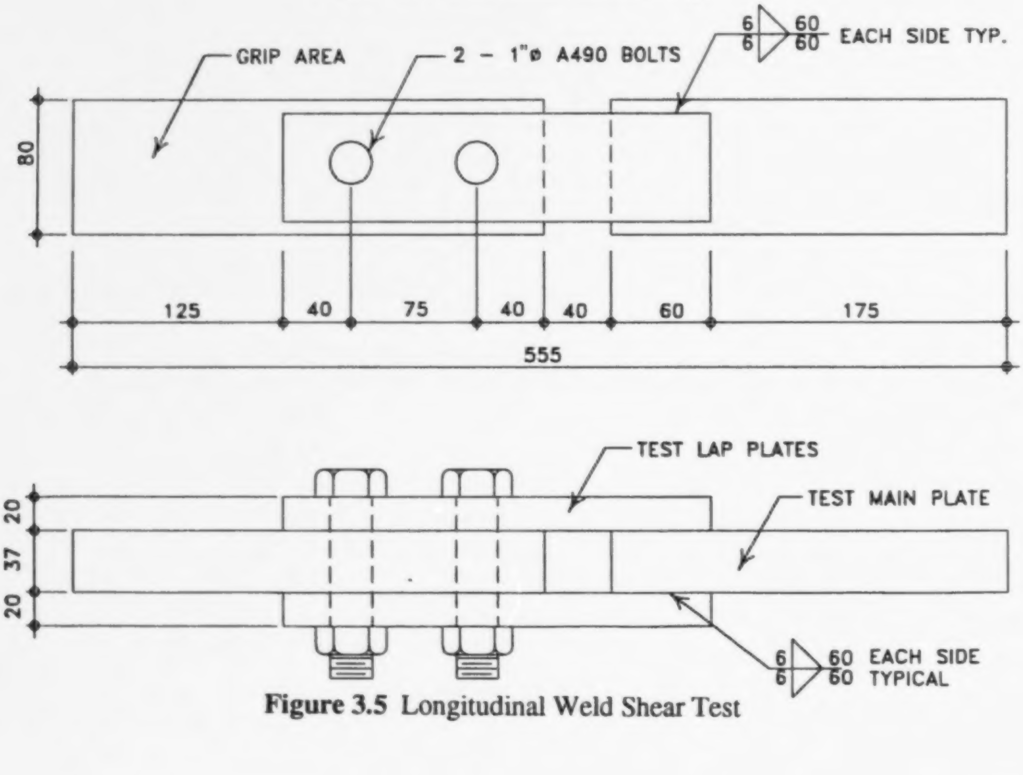

Figure 3.5 Longitudinal Weld Shear Test

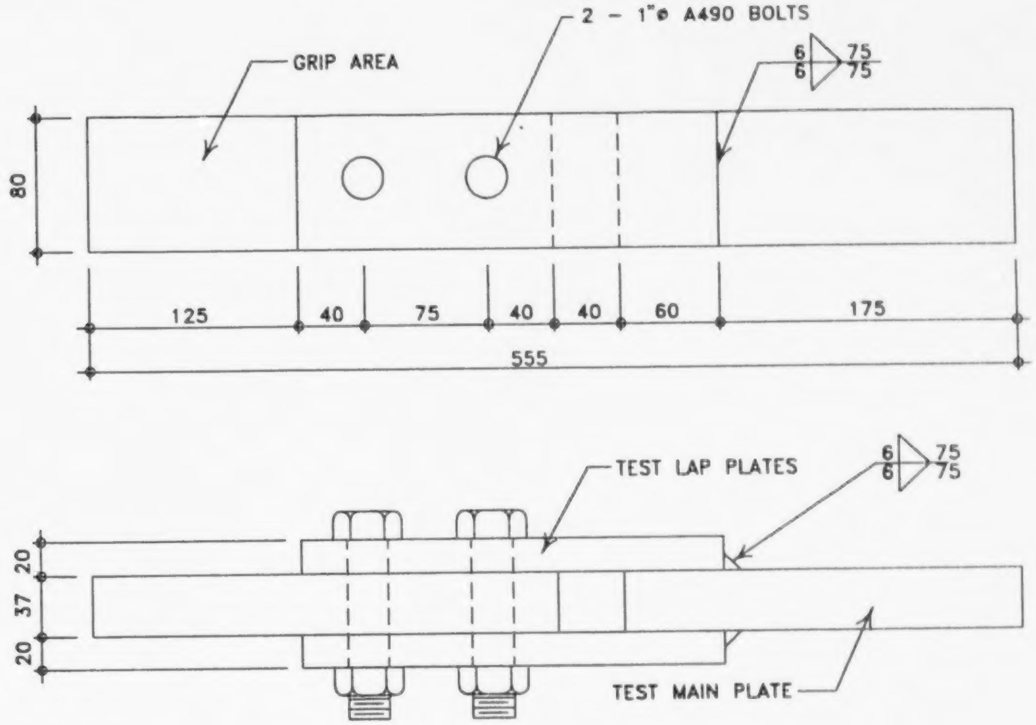

Figure 3.6 Transverse Weld Shear Test

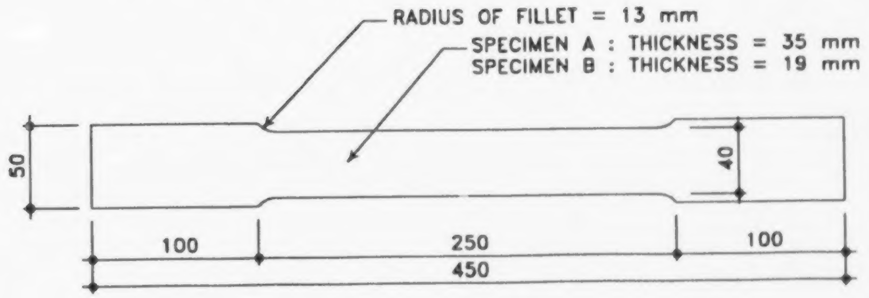
RADIUS OF FILLET = 13 mm
SPECIMEN A : THICKNESS = 35 mm
SPECIMEN B : THICKNESS = 19 mm
50
40
100
250
100
450

Figure 3.7 Steel Tension Coupon Test

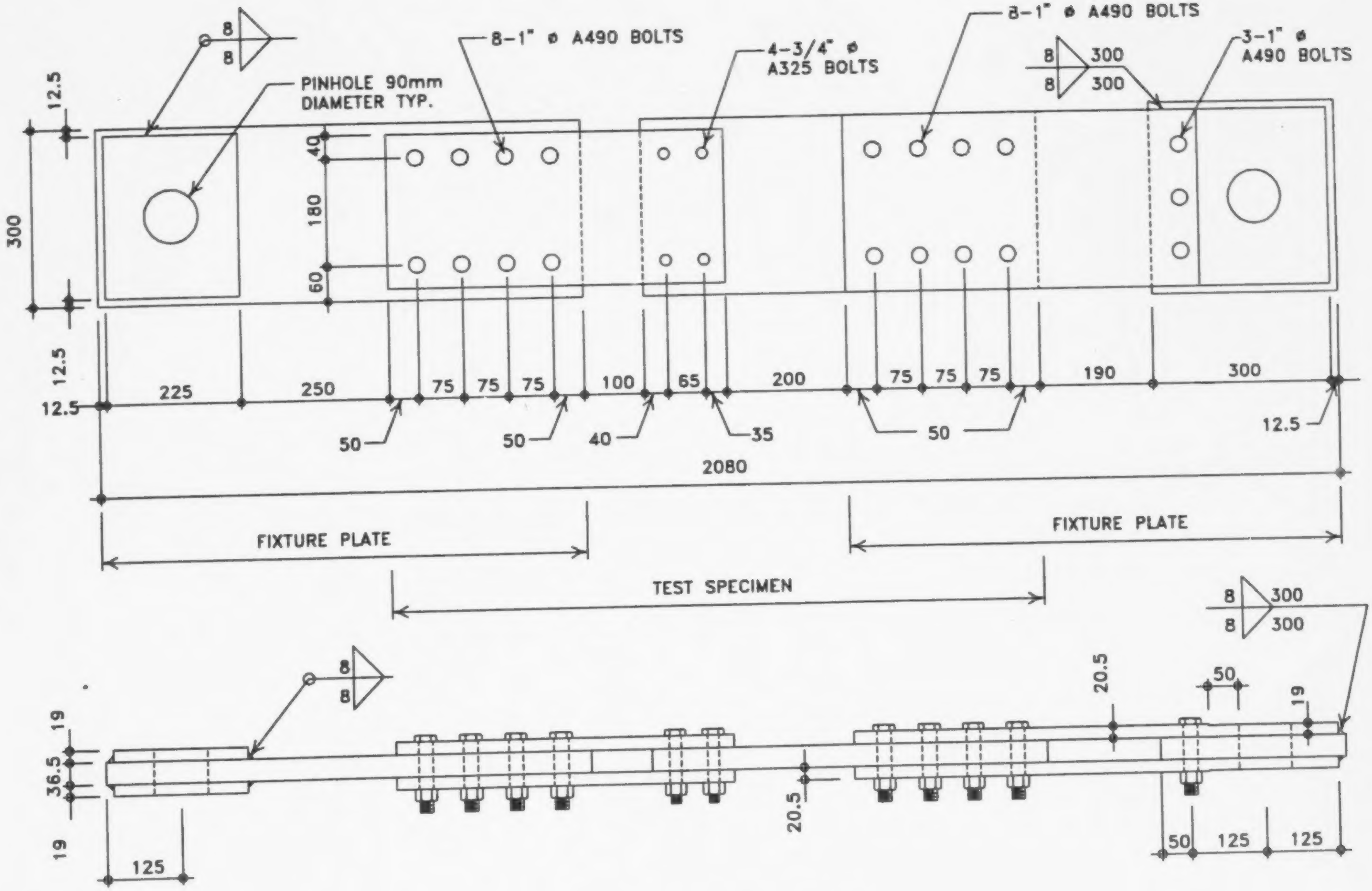

Figure 3.8 Main Test Set-Up

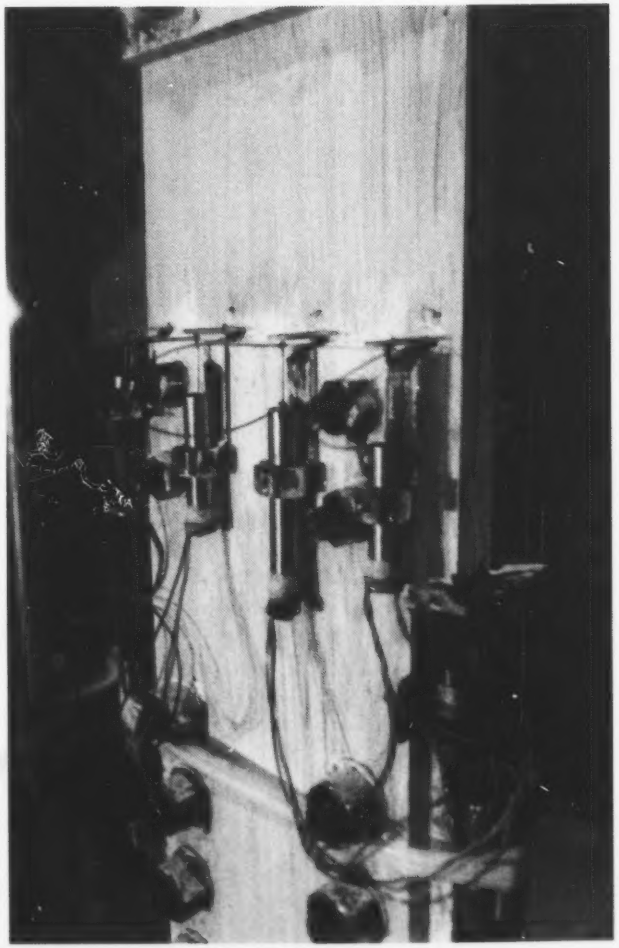

Figure 3.9 Main Test with All-Around Weld

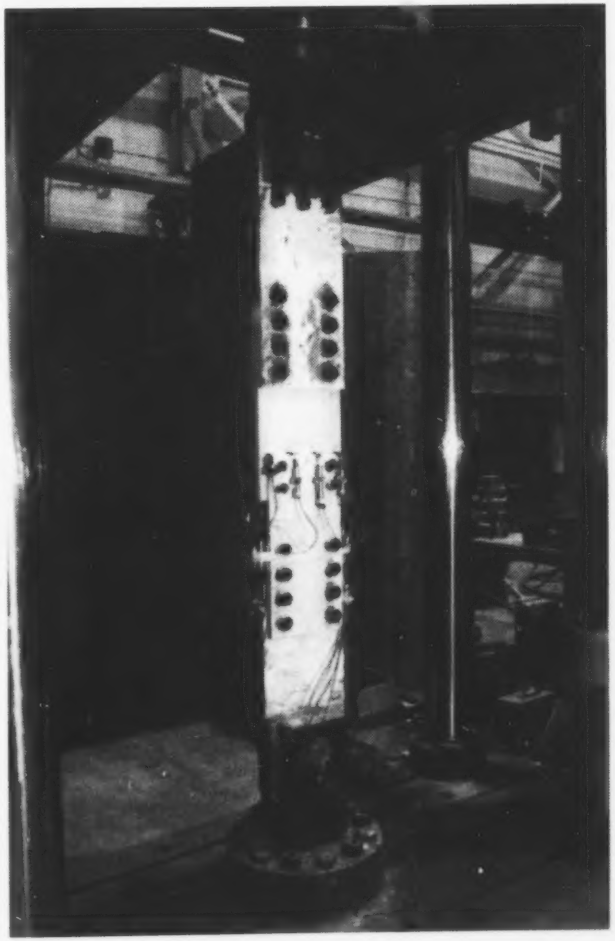

Figure 3.10 Main Test Overall View

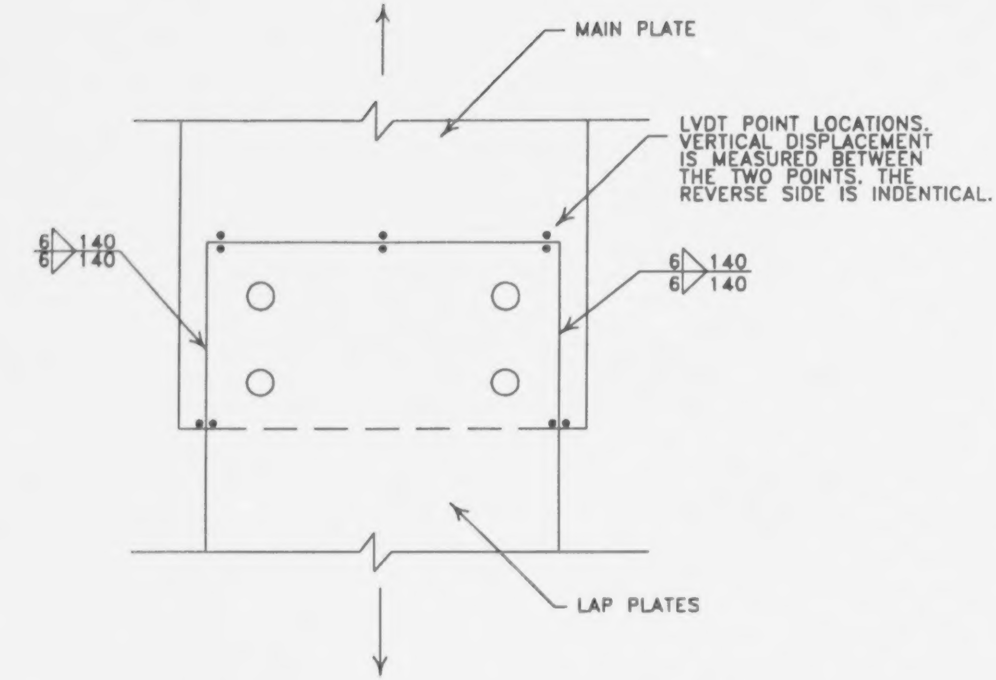
MAIN PLATE
LVDT POINT LOCATIONS. VERTICAL DISPLACEMENT IS MEASURED BETWEEN THE TWO POINTS. THE REVERSE SIDE IS INDENTICAL.
6 140
6 140
6 140
6 140
LAP PLATES

Figure 3.11 LVDT Locations for Full-Scale Specimens with Only Longitudinal Welds

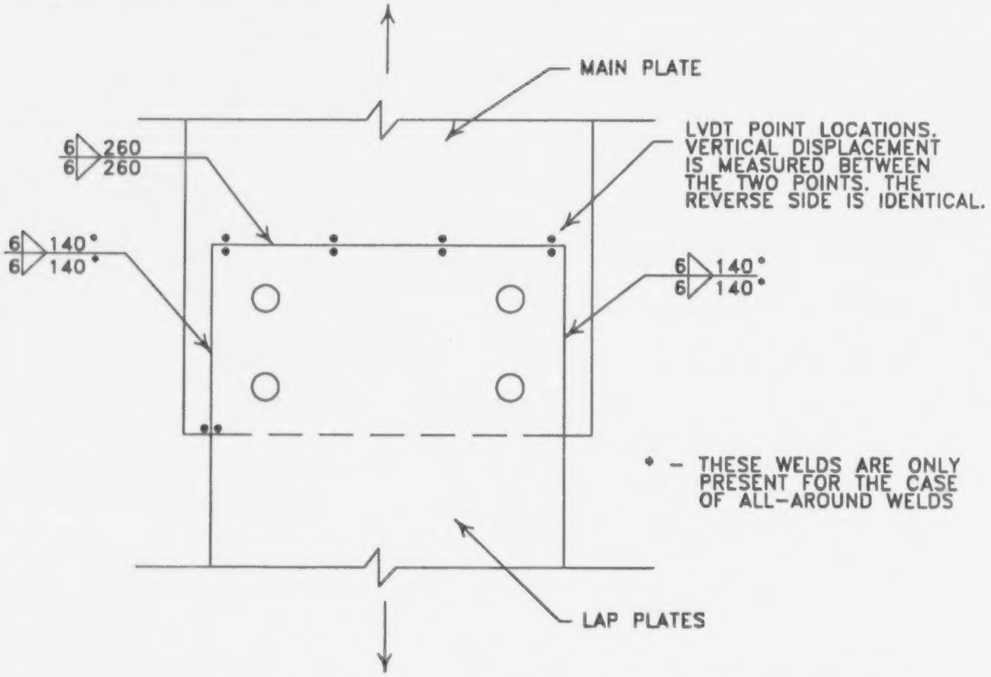
MAIN PLATE
LVDT POINT LOCATIONS. VERTICAL DISPLACEMENT IS MEASURED BETWEEN THE TWO POINTS. THE REVERSE SIDE IS IDENTICAL.
6 260
6 260
6 140*
6 140*
6 140*
6 140*
* – THESE WELDS ARE ONLY PRESENT FOR THE CASE OF ALL-AROUND WELDS
LAP PLATES

Figure 3.12 LVDT Locations for Specimens with Transverse Welds

4. TEST RESULTS

4.1 Ancillary Tests

4.1.1 General

The ancillary tests were vital to this research project; data from the bolt shear tests and the weld shear tests are used in later chapters to develop load versus deformation response equations. In turn, these equations are used for predicting the behaviour of the full-scale combination joints tested subsequently in the main testing program. It is also important to verify that the steel, the bolts, and the welds used are within nominal strength values. The data from each test reported herein are compared to existing experimental data from tests done by other researchers.

4.1.2 Bolt Tension Tests

Three 3/4 in. diameter bolts of ASTM grade A325 were tested in direct tension using a compression jig. In accordance with the testing procedure recommended in ASTM Standard A325-92a [ASTM 1992], each bolt was first subjected to a proof load of approximately 126 kN. No permanent deformation was noticed in any of the three specimens after the application of the proof load. The test data obtained showed that the average values for ultimate strength, ultimate deformation, and deformation at fracture were 218 kN, 3.8 mm, and 5.4 mm, respectively. According to ASTM Standard A325–92a, the specified minimum tensile strength must be at least 178 kN. Each of the three individual tests exceeded this value. Load versus elongation plots from the bolt tension tests are shown in Figure 4.1. The individual results are listed in Table 4.1.

The tensile stress area of a 3/4 in. diameter bolt in tension is equal to 215 mm^2. Thus, the average tensile strength (σ_u) of the bolts tested is 1012 MPa.

4.1.3 Bolt Shear Tests

Five 3/4 in. diameter bolts of ASTM grade A325 were tested in double shear using a tension jig. The shear characteristics of the bolts used in this test series are outlined in Table 4.2 and load versus deformation plots are shown in Figure 4.2. The average values of the ultimate shear resistance and the deformation at ultimate for these tests are 349 kN and 3.8 mm, respectively. A comparison of these results with those reported by others (see Chapter 2) is shown in Table 4.3.

Crawford and Kulak [1971] report approximately the same ultimate shear strength as identified in the tests described herein for bolts of ASTM grade A325 and 3/4 in. diameter. In comparing the Crawford and Kulak load versus deformation curve with curves reported herein (Figure 4.3), it is evident that the Crawford and Kulak results indicate that their bolts had considerably greater ductility. The difference in these two sets of results can be explained, at least in part. First, in the Crawford and Kulak tests the shear was induced using a compression jig, rather than a tension jig, as was used in these tests. Bolt shear and ductility results presented by Fisher [1965] indicate that, in addition to exhibiting higher strength, bolts tested using a compression jig rather than a tension jig tend to exhibit more ductility. Second, the Crawford and Kulak tests used thinner plates than used herein and the plate material had a nominal yield strength of only about 82 % of that used herein. Because the measured deformation includes shear deformation of the bolt, bending deformation of the bolt, and the localized deformation of the plate resulting from bearing, using thinner plates of lower strength will result in higher measured deformation. Notwithstanding these factors, the deformation results presented by Crawford and Kulak still seem rather high by comparison.

The test results presented by Wallaert and Fisher [1965] indicated an ultimate bolt shear strength that is also about the same as the ultimate strength found in the tests reported herein. Their data show the ductility of the bolts to be much less than that of

Crawford and Kulak [1971], but much closer to the experimental data presented herein. The ductility values presented by Wallaert and Fisher are slightly more than the ones presented here. However, again this can be explained by the fact that steel used for their tests was thinner and had a lower nominal yield strength than that used herein. Other differences in the Wallaert and Fisher study as compared with both of the other sets of data can be attributed to the fact that their data was from 7/8 in. diameter bolts, rather than from 3/4 in. diameter bolts.

According to the S16.1 Standard [CSA, 1994], the shear strength of a bolt is to be taken as 0.60 times its ultimate strength. The *Guide to Design Criteria for Bolted and Riveted Joints* [Kulak et al., 1987] suggests that 0.62 be used. The average shear strength of the bolts from the tests reported herein is 613 MPa, reflecting a ratio of shear strength to ultimate tensile strength of 0.61.

4.1.4 Weld Shear Tests

Weld shear tests were performed in order to determine the material properties of six millimeter leg size fillet welds deposited using E48018-1 electrodes. In the first series of tests, five tension jigs, each with two transverse welds of 75 mm length, were tested. The second series of tests had four tension jigs, each with four longitudinal welds of 60 mm length. The material properties of the weld specimens found in these series are outlined in Table 4.4 and Table 4.5. To make it easier to compare the research reported herein to research done by others, the weld load resistance values have all been normalized. Thus, all values of weld load resistance given following are presented using dimensions of load per millimeter of weld leg size per millimeter of weld length. Although some of the reported tests used E410xx grade electrodes rather than E480xx grade electrodes, the values of weld load resistance have not been normalized with respect to electrode strength. Load versus deformation plots from the tests are shown in Figure 4.4 and Figure 4.5.

For the longitudinal weld shear tests, the average normalized maximum shear resistance and average maximum deformation were 0.348 kN/mm/mm and 1.2 mm, respectively. For transverse welds, the average normalized maximum shear resistance and average maximum deformation were 0.458 kN/mm/mm and 0.52 mm, respectively. A comparison of these values with the values reported by Lesik and Kennedy [1988] and the values reported by Butler et al. [1972] is shown in Table 4.6

It is known that the angle at which the load is applied to the weld is an important factor in determining the load versus deformation characteristics of welds in shear. There are other factors involved as well, however. For example, the weld leg size, the length of weld used in the test program, and the nominal strength of the weld electrode used for depositing the weld can each have an effect on the weld ductility. Although all the test results shown here are reported using nominal resistances, each reported test had a unique set of test variables that could affect the load versus deformation characteristics. The tests reported by Butler et al. [1972], for instance, used a weld length of one inch (25 mm), whereas the rest of the reported tests use weld lengths in excess of 75 mm. When comparing these results, it is expected that there will be a certain amount of variance in the reported values.

Each of the three test programs give approximately the same values for the ultimate shear resistance of the welds for each of the transversely oriented case and the longitudinally oriented case. However, the data from the tests reported by Butler et. al [1972] reflects a lower shear strength for the welds as compared to the other two test programs. This was expected because the values used by Butler were from welds deposited using E60xx (now E410xx), while the remainder of the reported tests had weld deposited using E480xx electrodes. To compare the Butler values to the others, they can be adjusted to reflect the differing electrode strength by using the ratio of nominal strengths, that is, 480/410. Making this adjustment produces values of longitudinal and transverse ultimate weld strength of 0.317 and 0.460 kN/mm/mm, respectively. These adjusted values are closer to the average values of the other two tests.

Using the longitudinal weld values adjusted for weld electrode grade, the average normalized shear strength from the three sets of reported data is close to 0.30 kN/mm/mm; for transverse welds it is close to 0.45 kN/mm/mm. The design rules for weld shear strength given in the S16.1 design standard [CSA, 1994] reflect a transverse weld strength that is 1.5 times stronger than a longitudinal weld. The data from these sources support this: the ratios of transverse weld strength to longitudinal weld strength in the tests listed in Table 4.6 range from 1.3 to 1.5.

Although the maximum shear strengths found in these three sources of data were approximately the same in each case, the ultimate deformations that were reported were quite different. For the case of longitudinal welds (Table 4.6), the ultimate deformations ranged from 1.0 mm to 2.6 mm. The ultimate deformation reported by Lesik and Kennedy [1988] was approximately the same as the ultimate deformation found in the tests reported herein. The ultimate deformation reported by Butler et al. [1972], on the other hand, was over twice that reported in the other two tests. This may be as a result of the fact that the Butler tests used weld electrodes with a lower ultimate strength rating, thereby showing a different ductility. Unfortunately, measured values for the ultimate tensile strength of the deposited weld metal are not available for any of the three cases being examined.

For the case of transverse welds (Table 4.6), the values of ultimate deformation reported by Butler et al. [1972] were very close to the values from the tests reported herein, but the values reported by Lesik and Kennedy [1988] were quite low by comparison. As was the case in the longitudinal tests, the ultimate deformation values reported by Butler et al. were the greatest of the three sets of data.

4.1.5 Steel Tension Coupon Tests

The material properties found from the six tension coupons of this series are presented in Table 4.7. Steel coupons ST1, ST2, and ST3 were fabricated from the main plate steel of the full-scale testing program (35 mm thickness), and coupons ST4, ST5, and ST6 were fabricated from the lap plate steel of the full-scale testing program (19 mm thickness). The average modulus of elasticity for the main plate steel was 202 300 MPa and for the lap plate steel it was 204 500 MPa. The main plate steel displayed average static yield strength and average dynamic ultimate strength values of 348 MPa and 496 MPa, respectively. For the lap plate steel, the corresponding values were 285 MPa and 491 MPa, respectively. All of these experimental values are within the expected range for CAN/CSA-G40.21-M92 grade 300W steel.

Load versus deformation plots for the six steel tension coupon tests are shown in Figure 4.6 and Figure 4.7. (The dips in these curves represent the readings of static load.) While the coupon was still in the elastic range, there was good correlation between the data obtained using the extensometer and the data obtained using the readings from the LVDT's. When the deformation became too large to be recorded by either the LVDT's or the extensometer, readings taken from the stroke of the testing machine were used.

4.2 Results of Full-Scale Testing Program

4.2.1 General

The main testing program consisted of 24 full-scale double lap joints that were tested in tension. The full-scale tests are divided into three main groups: specimens with longitudinal welds in combination with high-strength bolts, specimens with transverse welds in combination with high-strength bolts, and specimens with both longitudinal and transverse welds in combination with high-strength bolts. Each of the three groups are subsequently divided into two sub-groups: specimens with preloaded bolts, and specimens in which the bolts were installed only to the snug-tight condition. Further, each of these sub-groups are divided into two series: specimens with bolts in a positive bearing condition at the start of loading, and specimens with bolts in a negative bearing condition at the start of loading. Two identical specimens were tested in each of these series. Thus, the 24 full-scale double lap joints represent a total of 12 different types of specimens.

Results from each of the 24 tests are outlined in Table 4.8, Table 4.9, and Table 4.10. It should be noted that for all 12 different series, the results of the two identical tests were quite close in value. Also, the three tables give load and deformation values for weld failure and bolt failure. In some cases, the maximum load in the connection was reached after the welds failed, because the bolts were proportioned to carry more load than the welds alone. For the purposes of this research, however, failure of the joint will be considered to be that at first failure of any mechanical fastener, which is always at failure of the welds. In each of the three main groups, at least five sets of comparisons can be made:

1. Within the category of specimens in a negative bearing condition, a comparison can be made between the results of tests using bolts tightened only to a snug-tight condition, and the results of tests using preloaded bolts. The difference in ultimate load between these two cases gives an indication of the amount of friction contribution in the negative bearing category.
2. Within the category of specimens in a positive bearing condition, a comparison can also be made between the results of tests using bolts tightened only to a snug-tight condition, and the results of tests using preloaded bolts. The difference in ultimate load between these two cases gives an indication of the amount of friction contribution in the positive bearing condition.
3. Considering all the specimens with only snug-tightened bolts in a particular

group, a comparison can be made between the test results from specimens in negative bearing with test results from specimens in positive bearing. This comparison gives an indication of the amount of bolt contribution that can be attributed to the positive bearing condition.

4. Considering all the specimens with preloaded bolts in a particular group, another comparison can also be made between the test results from specimens in negative bearing with test results from specimens in positive bearing. This comparison gives an indication of the amount of the bolt contribution that results from a positive bearing condition.
5. Considering all of the test specimens, the effects of preloaded bolts and the effects of positive bearing condition can be compared between each of the three main test groups (longitudinal welds only, transverse welds only, and both longitudinal and transverse welds).

In the following three sections, the first four of these comparisons will be made between the test results of the specimens with longitudinal welds, specimens with transverse welds, and specimens with both types of welds. Subsequently, there will be discussion regarding the results of the three different main test groups. All comparisons will be made based on the values of load and deformation at the time of weld failure.

4.2.2 Specimens with Longitudinal Welds in Combination with Bolts

Of the three main groups of full-scale test specimens, the group containing longitudinal welds acting in combination with high-strength bolts behaved in the most predictable manner. In Table 4.8, specimens in the "N" sub-group are those in which the bolts were in a negative bearing condition, and those in the "P" sub-group are those in which the bolts were in a positive bearing condition at the start of loading. Within each of these two categories, two of the four specimens are in the “S” series, in which the bolts are tightened only to the snug-tight condition, and the other two specimens are in the "P" series, in which the bolts are preloaded. All of the specimens reported in Table 4.8 belong to the "L" group, which contains specimens with welds oriented longitudinally to the direction of loading.

The first comparison that will be made is between the NSL series and the NPL series. The only difference between the specimens of these two series was the tension in the bolts of the specimens. The bolts in the specimens of the former test series were installed only to the snug-tight condition, and the bolts in the specimens of the latter test series were preloaded. On average, an increase in load resistance at weld failure of 470 kN was obtained in the preloaded bolt tests as compared with the non-preloaded case (NPL vs. NSL), which is an increase in strength of 18 %. It is reasonable to attribute this increase to friction between the connected parts.

The second comparison is between the PSL and the PPL series (bolts in positive bearing). As with the first comparison, the only difference in these specimens is that the bolt tensions in the two groups were different. The PPL series gave results that were, on average, 118 kN stronger than the PSL series, which is a five percent increase in strength. This increase is modest, and the variation within the PSL series alone is 145 kN. The 118 kN increase for this case may be just experimental variation. Further analysis is needed to determine if it is reasonable to attribute this increase to friction between the parts.

The third comparison that can be made is between the results from test specimens in the NSL series and results from the PSL series (snug-tight bolts). These two sets of tests differed from each other only in the condition of bearing at the start of loading. The NSL series has bolts in the negative bearing condition and the PSL series has bolts in the positive bearing condition. The PSL series specimens were on average 1035 kN, or 81 %, stronger than the specimens from the NSL series.

The fourth comparison is made between the results of the specimens of the NPL series and the PPL series (preloaded bolts). As was the case with the third comparison, the only difference between the two tests was the condition of bearing at the start of loading. The ultimate load resistance found in the PPL series test specimens was on average 682 kN, or 39%, stronger than the load resistance found in the NPL series.

The importance of the bearing condition of the bolts in a combination joint is emphasized through these comparisons. As was anticipated, the ultimate strength was affected by the bearing condition of the bolts, but the tests also showed that the ductility of the joints was affected. The information shown in Table 4.8 indicates that joints with bolts in positive bearing from the start of loading showed slightly more deformation at weld failure than those with bolts in negative bearing at the start of loading. For example, the deformation at maximum load for tests in the PPL series was an average of 3.3 mm, but for the NPL series it was 2.5 mm. As has already been described, the amount of tension in the bolts did affect the ultimate strength, but the ductility of these four types of joints was not significantly affected by this factor.

Specimens with preloaded bolts displayed more ductility than those with only snug-tightened bolts in the negative bearing case. They displayed less ductility in the positive bearing case. The differences were not significant in either category, however. Weld failure in this group occurred predominately through the throat of the weld, that is, at 45° to the weld leg.

It has already been indicated that the failure load of the combination joints will be defined as that corresponding to fracture of the first elements, which is always the welds. As seen in Table 4.8, this failure load may or may not be the ultimate load the connection resists. In two of the eight test specimens, the load at which the bolts failed was greater than that at which the welds failed. In the last four specimens of Table 4.8, the load at weld failure is the same as the load at bolt failure. This indicates a simultaneous failure of the bolts and the welds.

4.2.3 Specimens with Transverse Welds in Combination with Bolts

The second major group of full-scale tests was comprised of test specimens combining preloaded high-strength bolts with welds oriented transverse to the direction of load. The overall load versus deformation behaviour of these connections was more difficult to predict than that of the first group because the welds in these tests have limited ductility relative to the bolts. Also as a result of the low relative ductility, the portion of the total load contributed by the bolts at the time of weld failure is quite low. As was the case with the first test group, this group showed a wide range of ultimate loads (see Table 4.9). The following paragraphs parallel the four comparisons presented in Section 4.2.2 for combination joints that had longitudinal welds. (The nomenclature of the joints was given in Section 3.4.2)

The first comparison is between specimens in the NST and NPT series of tests; both had bolts in the negative bearing condition at the start of loading. The NST series had bolts tightened only to a snug-tight condition, however, and the NPT series had preloaded bolts. The effect of preloading is more prominent in this group of test specimens than it was when the welds were longitudinal (Section 4.2.2). The NPT series results showed an ultimate load resistance that was an average of 291 kN more than the NST series, which is a 21 % increase in ultimate strength.

In the second comparison, test results from the PST and PPT series are reviewed. Both types of specimens have a positive bearing condition at the start of loading, but the latter has preloaded bolts and the former has bolts tightened only to a snug-tight condition. The tests results from the PPT series indicates an average ultimate strength that is 509 kN greater than that indicated by the PST series test results. This is a 33 % increase, which can be attributed to the friction in the plates.

The third comparison that can be made is between the results of the NST test results and the PST test results (snug-tight bolts). The NST series test specimens were in a negative bearing condition, and the PST series test specimens were in a positive bearing condition. The PST series shows an ultimate load resistance that was an average of 139 kN greater than that of the NST series, which is a 10 % increase in ultimate load as a result of positive bearing versus negative bearing.

In the fourth comparison, the NPT series versus PPT series, the bolts were preloaded in both cases. As with the previous comparison, the difference between the two sets of data was that the former was in a negative bearing condition and the latter was in a positive bearing condition. The comparison shows a 358 kN difference in average ultimate load resistance, reflecting a 21 % increase in strength as a result of having a positive bearing condition.

The deformations at weld failure in this test group were generally quite consistent. Aside from one of the tests, which had a deformation of 0.20 mm, the recorded deformations ranged from 0.33 mm to 0.44 mm. This is an acceptable range considering that the ultimate failure deformation of the entire joint, or the deformation when the bolts fail, is typically in excess of four millimeters. Fractures of the welds in this testing series were located more or less in the plane of the main plate, that is, along the weld leg in the same plane as the line of force.

The results shown in Table 4.9 indicate that out of the eight test specimens of this group, only two had simultaneous failure of bolts and welds. In all six of the other tests, the welds fractured first, followed by the bolts. In this group, the ultimate load of the connection was always attained at the time of weld fracture.

4.2.4 Specimens with All-Around Welds in Combination with Bolts

The last major group of full-scale specimens to be discussed is that in which both longitudinal and transverse welds were present in combination with high-strength bolts. It appears that very little, if any, load-sharing had taken place between the bolts and the welds in this configuration. In the following paragraphs, the comparisons of values from Table 4.10 will again parallel those made in Section 4.2.2.

The first comparison will be made between the NSA series and the NPA series (negative bearing condition). The specimens in the NSA series had bolts tightened only to the snug-tight condition, and the specimens in the NPA series had preloaded bolts. There was a 196 kN increase in load resistance in the NPA series as compared to the NSA series test results. Thus, as a result of the preloading of the bolts, there was an increase in strength of nine percent.

The results from the PSA series and the results from the PPA series (positive bearing condition) will be compared next. As in the previous case, these tests differed only in that the tension in the bolts differed. The test specimens in the PPA series had an average ultimate load resistance that was 372 kN greater than that of the PSA series, which is a 15 % increase in strength. Again, it is reasonable to attribute this increase to frictional resistance between the component plates.

In the third comparison, results from the NSA series of tests are compared to results from the PSA series; both have bolts tightened only the snug-tight condition. The bolts in the specimens of the NSA series are in a negative bearing condition at the start of loading, and the bolts of the specimens in the PSA series are in a positive bearing condition at the start of loading. A comparison of the average ultimate load resistance of the tests in the NSA series with that of the NPA series tests shows an increase of 295 kN, or 13 %, as a result of the bearing condition of the bolts.

The fourth, and last, comparison is between the NPA series test results, and the PPA series test results (preloaded bolts). These specimens differed in their bearing condition at the start of loading; the NPA series had negative bearing, and the PPA series had positive bearing. On average, the ultimate load resistance from the PPA series was 471 kN greater than that of the NPA series, corresponding to a 20 % increase in strength as a result of the bearing condition of the bolts.

In this group, the deformation at the time of weld failure varied widely. It ranged from 0.27 mm to 1.2 mm. With three different types of mechanical fasteners (bolts, transverse welds, and longitudinal welds), each with entirely different ductilities, it is quite difficult to accurately predict the joint deformation. As was the case with the other two test groups, the longitudinal welds fractured along the weld throat and the transverse welds fractured through the weld leg parallel to the main plate.

4.3 Discussion of Full-Scale Test Results

4.3.1 Effect of Preloading of Bolts

Comparison numbers one and two, as outlined in Section 4.2.1, compare the test results from specimens that are identical except for the tension in their bolts. These comparisons are made in Sections 4.2.2 through 4.2.4, using the test data outlined in Table 4.8 through Table 4.10.

It is expected that the load resistance due to friction in slip-critical connections can only be relied upon up to the service load level. According to the S16.1 Standard [CSA, 1994], at ultimate load, the bearing resistance of the slip-critical connection must equal or exceed the factored load because it is anticipated that the plate friction cannot be expected to be present as the ultimate load is reached. Although this is reasonable for connections that possess only bolts, the results of the full-scale tests reported herein support the notion that plate friction in combination joints contributes to the ultimate strength. A noticeable increase in ultimate load resistance was observed in tests with preloaded bolts as compared to the tests that had bolts tightened only to a snug-tight condition. Thus, friction resulting from the clamping force of the bolts is still present and acts in combination with the shear resistance of the welds at the time the ultimate load level is reached (weld fracture).

The amount of resistance contributed by friction at the ultimate load level was not particularly consistent with each group, however. The strength increases attributed to plate friction ranged from 118 kN to 509 kN. Furthermore, there did not seem to be any discernible trend related to the configuration of the welds and the plate friction contribution. For example, in the negative bearing condition, joints with longitudinal welds showed the most friction load resistance, but for the positive bearing condition joints with longitudinal welds showed the least amount of friction load resistance.

4.3.2 Effect of Bearing Condition of Bolts

What was anticipated, and observed, was that at fracture of the welds no load sharing occurs between welds and bolts in bearing when the bolts are in a negative bearing condition at the start of loading. The bolt hole clearance provided in the full-scale tests was a total of 2 mm. Thus, when the bolts are in a negative bearing condition at the start of loading, the connection plates must slip a distance of 4 mm with respect to each other before the bolts bear against the connection plates. Data from the ancillary tests indicate that welds oriented longitudinally to the direction of loading will fracture at an average deformation of 1.2 mm, and welds oriented transversely to the direction of loading will fracture at an average deformation of 0.52 mm. When a joint has deformed enough to allow the bolts to come into bearing, the welds in the connection will most likely have already failed. Thus, it is reasonable to assume that any increase in strength attained as a result of using a positive bearing condition rather than a negative bearing condition can be attributed to the shear resistance of the bolts as the connected parts come into contact with the bolt shank.

Comparison numbers three and four, as outlined in Section 4.2.1, relate the test results from specimens that are identical except for their bearing condition. Observations regarding these tests were in Sections 4.2.2 through 4.2.4 and the test data were outlined in Table 4.8 through Table 4.10. The results of these comparisons clearly show that specimens with a positive bearing condition at the start of loading are stronger than specimens with a negative bolt bearing condition at the start of loading. However, the increase in strength varies with the configuration of the combination joint.

In both sets of comparisons, specimens with snug-tight bolts and specimens with preloaded bolts, the same trend was observed. Insofar as being able to share load with bolts, the three weld configurations rank in the following order, from best to worst: longitudinal welds only, both longitudinal and transverse welds, and transverse welds only. This trend in ultimate strength indicate the importance of knowing the actual bearing condition of the bolts at the start of loading; there is a wide range of results that can occur. Also apparent from these comparisons is that the beneficial effects of the bearing condition of the bolts is diminished when the bolts are preloaded.

Table 4.1 Results of Bolt Tension Tests

Specimen	Ultimate Strength kN	Deformation at Ultimate mm	Deformation at Fracture mm
BTT-1	223	3.7	4.9
BTT-2	215	3.8	5.8
BTT-3	217	4.0	5.6

Table 4.2 Material Properties of Bolts in Shear

Specimen	Ultimate Shear Load Resistance kN	Maximum Shear Strength MPa	Deformation at Ultimate Load mm
BST1	341	598	3.9
BST2	343	602	3.6
BST3	360	632	3.9
BST4	362	635	3.7
BST5	340	596	4.0

Table 4.3 Comparison of Bolt Shear Properties

Source	Ultimate Shear Load Resistance kN	Deformation at Ultimate Load mm
Present Investigation	349	3.8
Crawford and Kulak [1971]	330	8.9
Wallaert and Fisher [1965]	314	4.9

Table 4.4 Transverse Weld Shear Test Results

Specimen	Ultimate Shear Resistance* kN/mm/mm	Deformation at Ultimate mm
TST1	0.448	0.35
TST2	0.471	0.70
TST3	0.516	0.51
TST4	0.428	0.49
TST5	0.475	0.55

*Normalized values

Table 4.5 Longitudinal Weld Shear Test Results

Specimen	Ultimate Shear Resistance* kN/mm/mm	Deformation at Ultimate mm
LST1	0.329	1.1
LST2	0.343	1.4
LST3	0.380	1.1
LST4	0.327	1.0

* Normalized Value

Table 4.6 Comparison of Weld Shear Properties

	Transverse Welds		Longitudinal Welds	
Source	Ultimate Shear Resistance kN/mm/mm	Deformation at Ultimate mm	Ultimate Shear Resistance kN/mm/mm	Deformation at Ultimate mm
Present Investigation	0.458	0.52	0.348	1.2
Lesik and Kennedy [1988]	0.437	0.30	0.292	1.0
Butler et al. [1972]	0.393	0.64	0.271	2.6

Table 4.7 Steel Tension Coupon Material Properties

Specimen	Modulus of Elasticity MPa	Static Yield Strength MPa	Static Ultimate Strength MPa	Dynamic Ultimate Strength MPa	Strain at Ultimate Strength %	Rupture Strain %
ST1	206 000	343	470	492	15.5	30.7
ST2	199 000	341	458	484	13.5	—
ST3	202 000	359	491	513	15.0	30.1
ST4	202 000	280	459	489	17.5	30.2
ST5	202 000	288	465	492	17.3	28.8
ST6	209 500	286	466	493	18.7	31.1

Table 4.8 Results of Longitudinal Weld Combination Joint Tests

Specimen Designation	Ave. Deform. at Weld Failure mm	Load at Weld Failure kN	Ave. Deform. at Bolt Failure mm	Load at Bolt Failure kN
NSL - 1	2.1	1308	5.6	1579
NSL - 2	1.1	1234	5.5	1513
NPL - 1	2.3	1776	6.7	1450
NPL - 2	2.6	1706	5.8	1641
PSL - 1	3.5	2233	3.5	2233
PSL - 2	3.9	2378	5.2	2378
PPL - 1	3.5	2418	3.5	2418
PPL - 2	3.1	2428	3.1	2428

Table 4.9 Results of Transverse Weld Combination Joint Tests

Specimen Designation	Ave. Deform. at Weld Failure mm	Load at Weld Failure kN	Ave. Deform. at Bolt Failure mm	Load at Bolt Failure kN
NST - 1	0.35	1380	6.0	1295
NST - 2	0.40	1400	5.4	976
NPT - 1	0.33	1676	6.9	1346
NPT - 2	0.20	1685	5.2	1182
PST - 1	0.38	1474	4.5	1375
PST - 2	0.44	1584	3.9	1359
PPT - 1	0.43	2111	0.43	2111
PPT - 2	0.44	1965	0.44	1965

Table 4.10 Results of All-Around Weld Combination Joint Tests

Specimen Designation	Ave. Deform. at Weld Failure mm	Load at Weld Failure kN	Ave. Deform. at Bolt Failure mm	Load at Bolt Failure kN
NSA - 1	0.44	2231	7.8	1416
NSA - 2	0.28	2195	6.4	1465
NPA - 1	0.27	2390	5.7	1640
NPA - 2	0.29	2428	4.8	1469
PSA - 1	1.2	2558	2.3	2399
PSA - 2	0.41	2458	2.1	2333
PPA - 1	0.50	2829	3.3	2641
PPA - 2	0.67	2930	2.4	2874

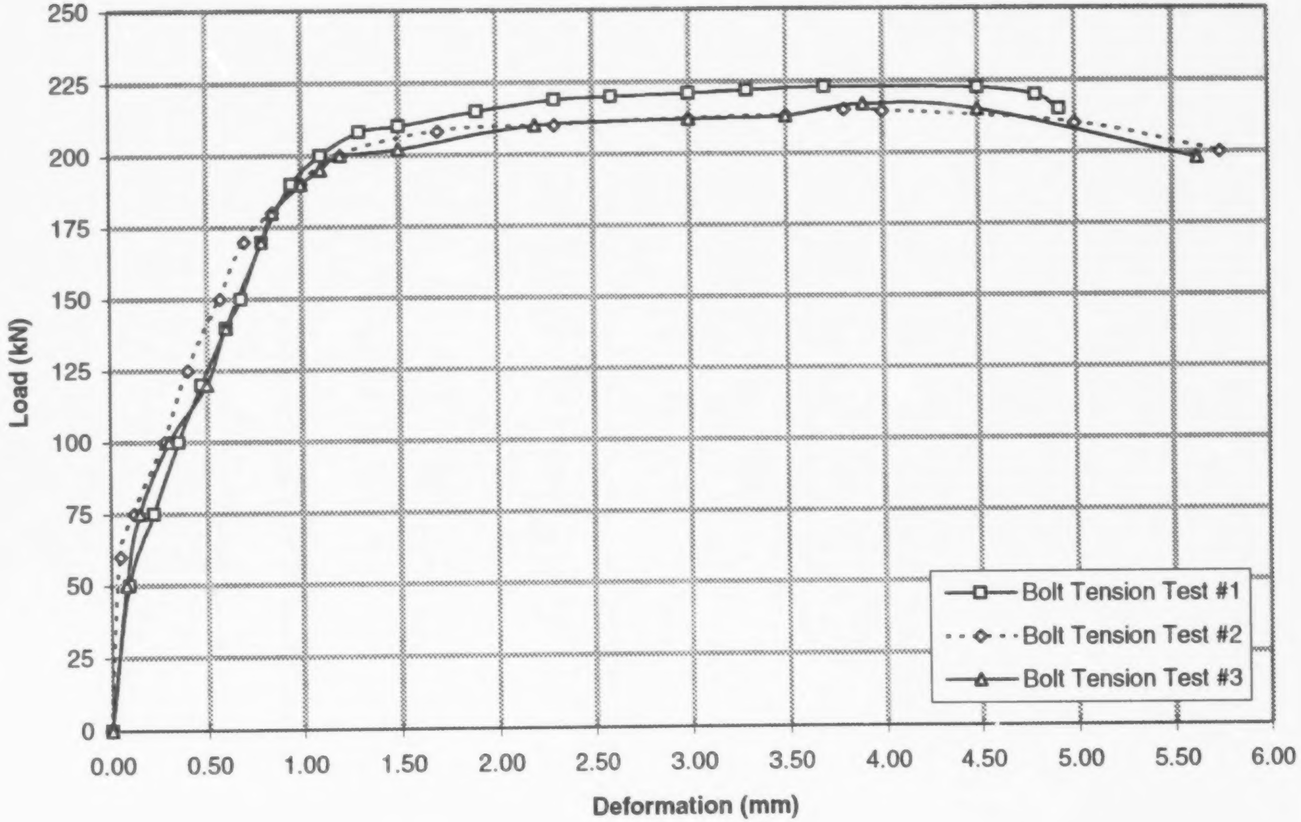

Figure 4.1 Load versus Elongation Plots for Bolt Tension Tests

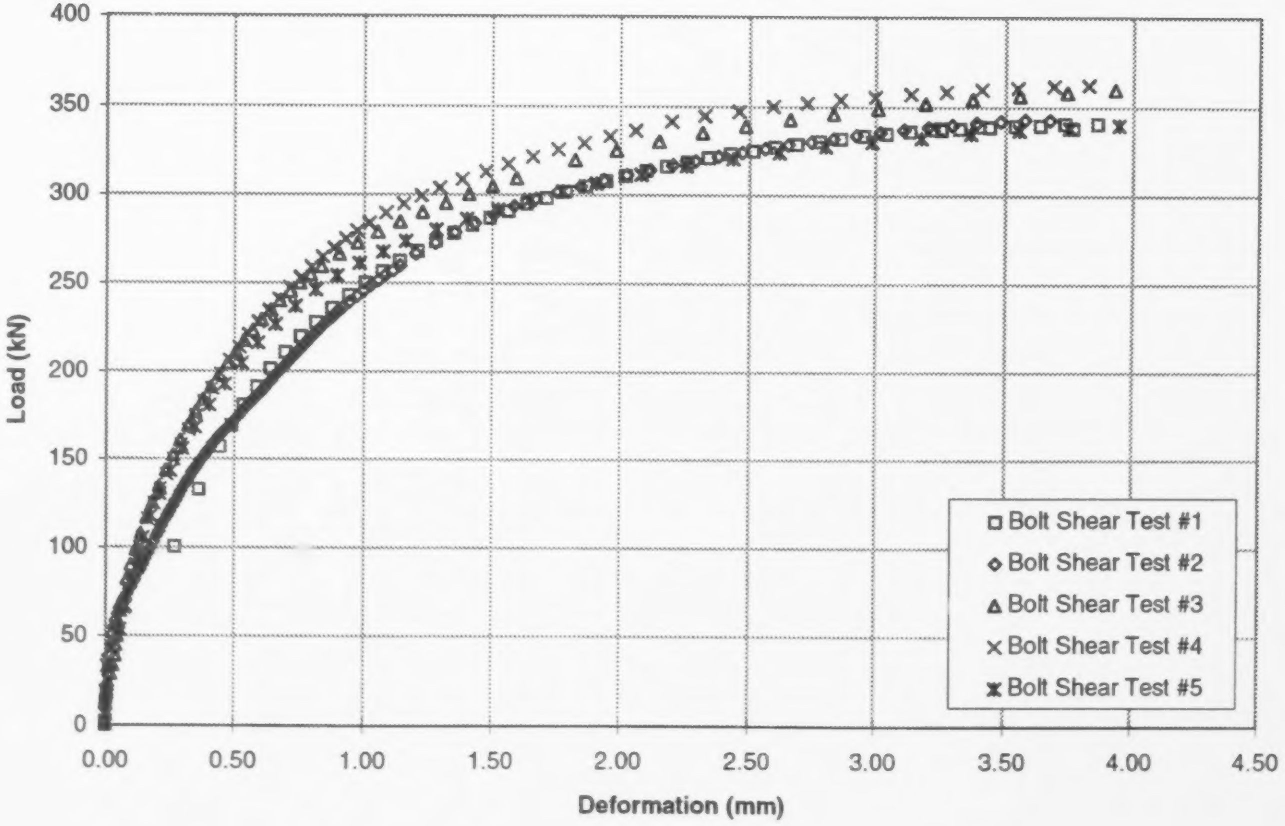

Figure 4.2 Load versus Deformation Plots for Bolt Shear Tests

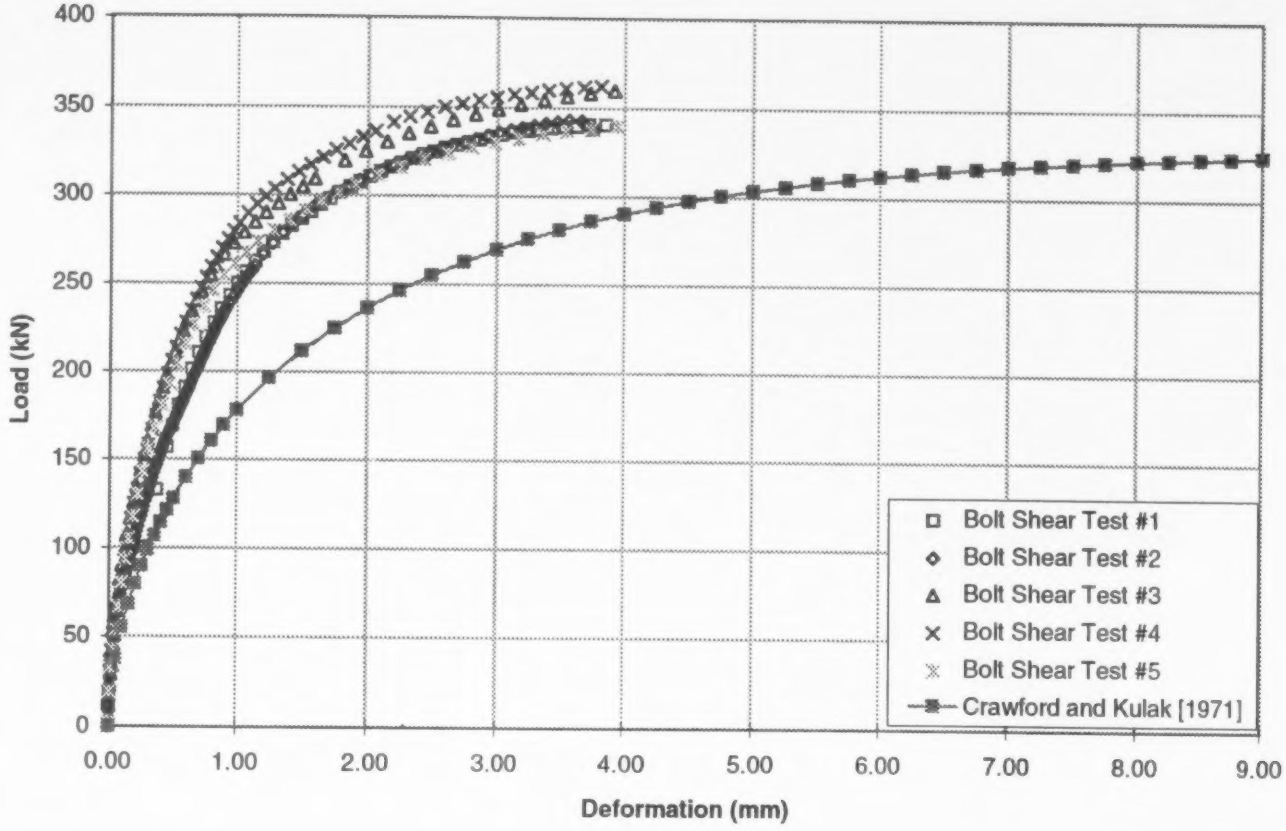

Figure 4.3 Comparison of Load versus Deformation Plots for Bolt Shear Tests

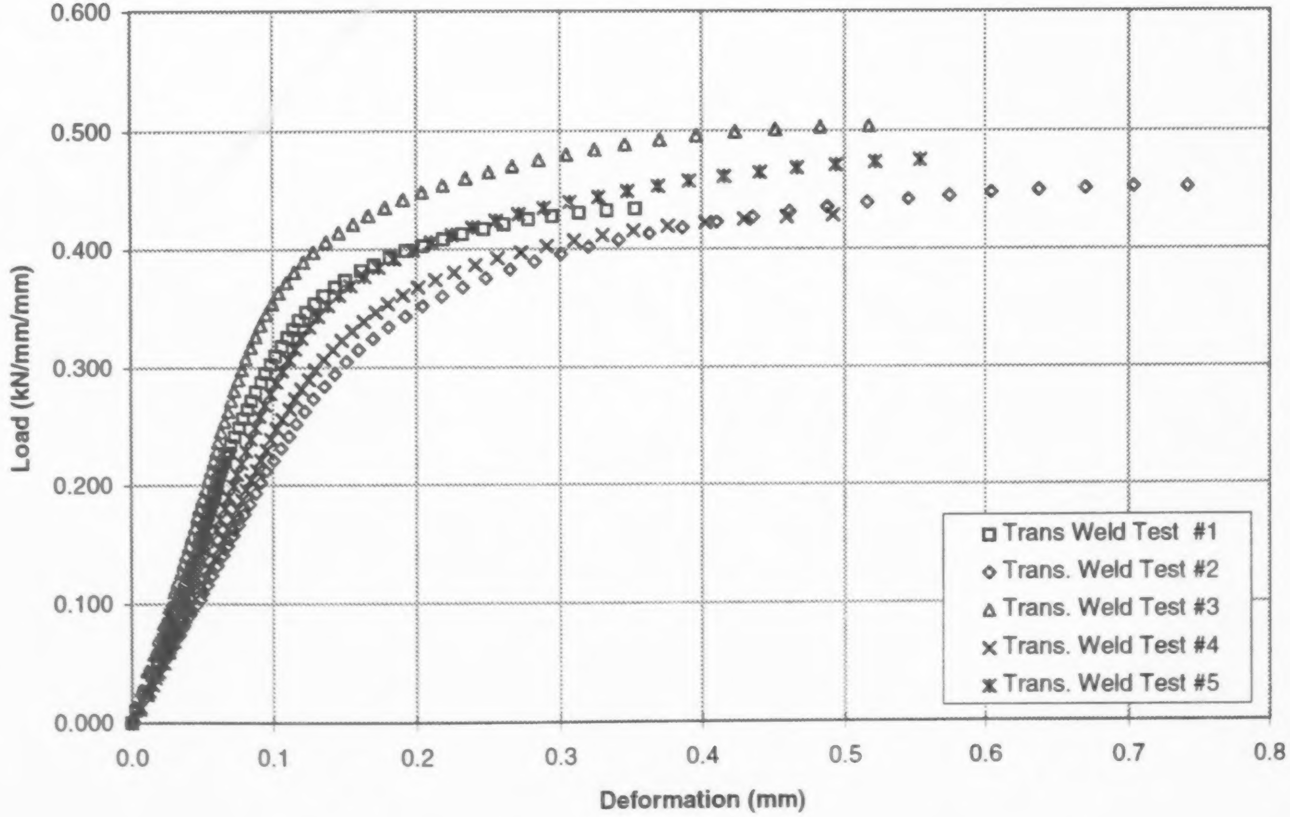

Figure 4.4 Load versus Deformation Plots for Transverse Weld Tests

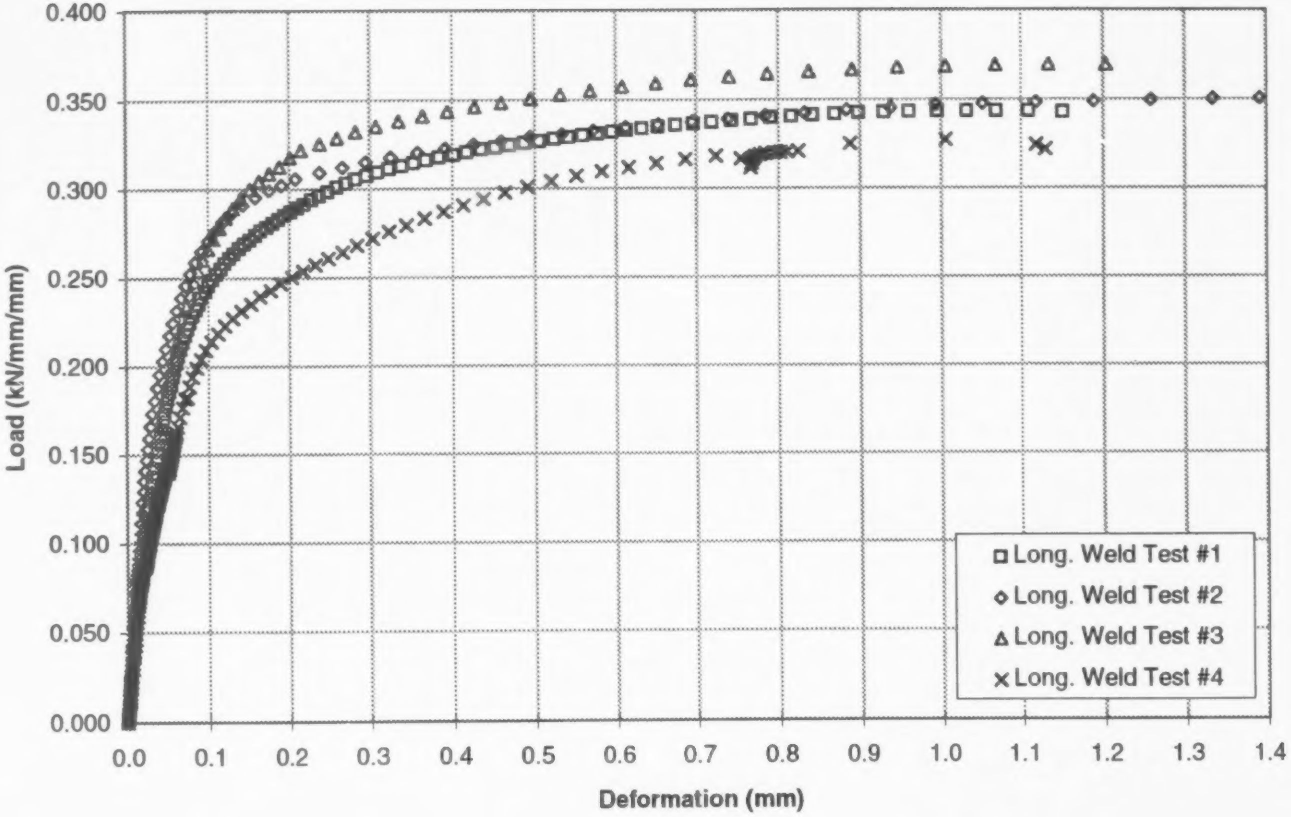

Figure 4.5 Load versus Deformation Plots for Longitudinal Weld Tests

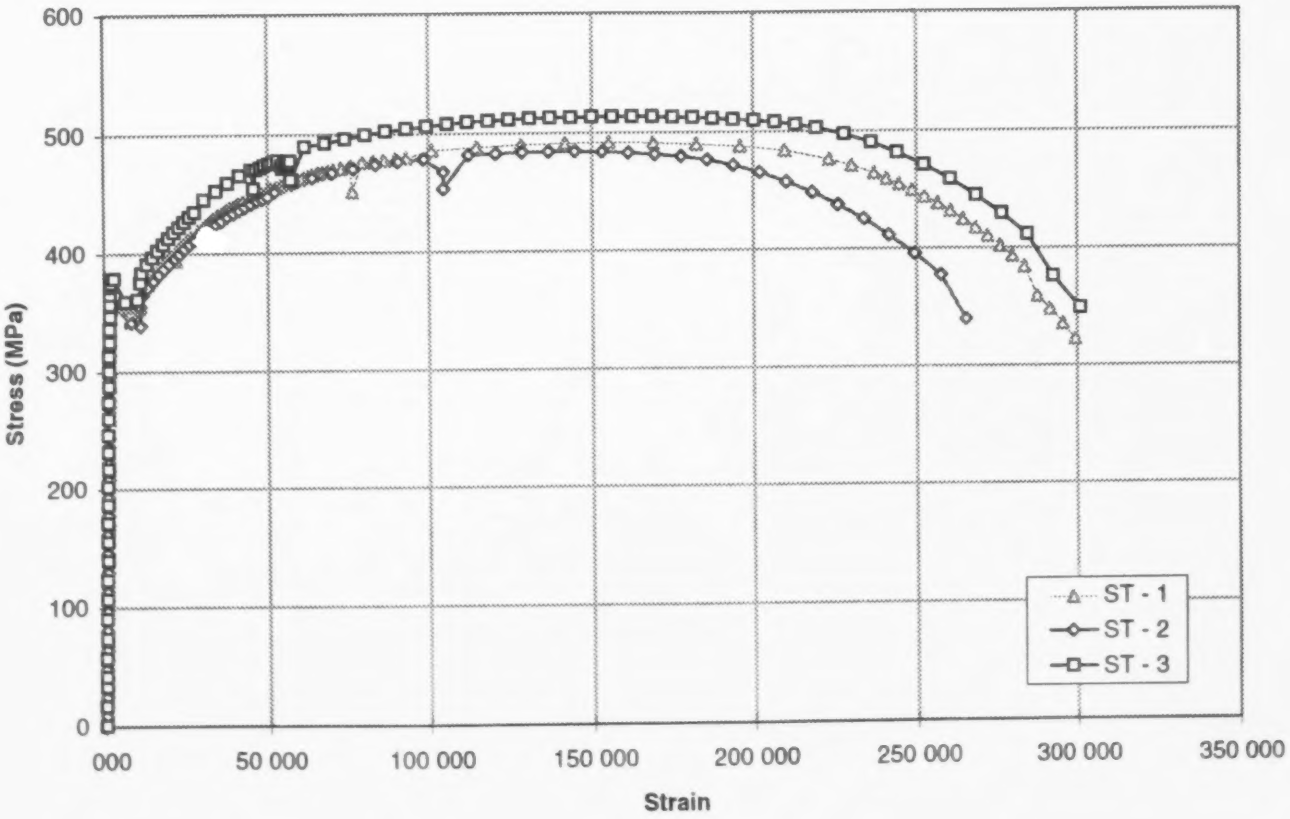

Figure 4.6 Stress versus Strain Plots for Tension Coupons of 35 mm Thickness

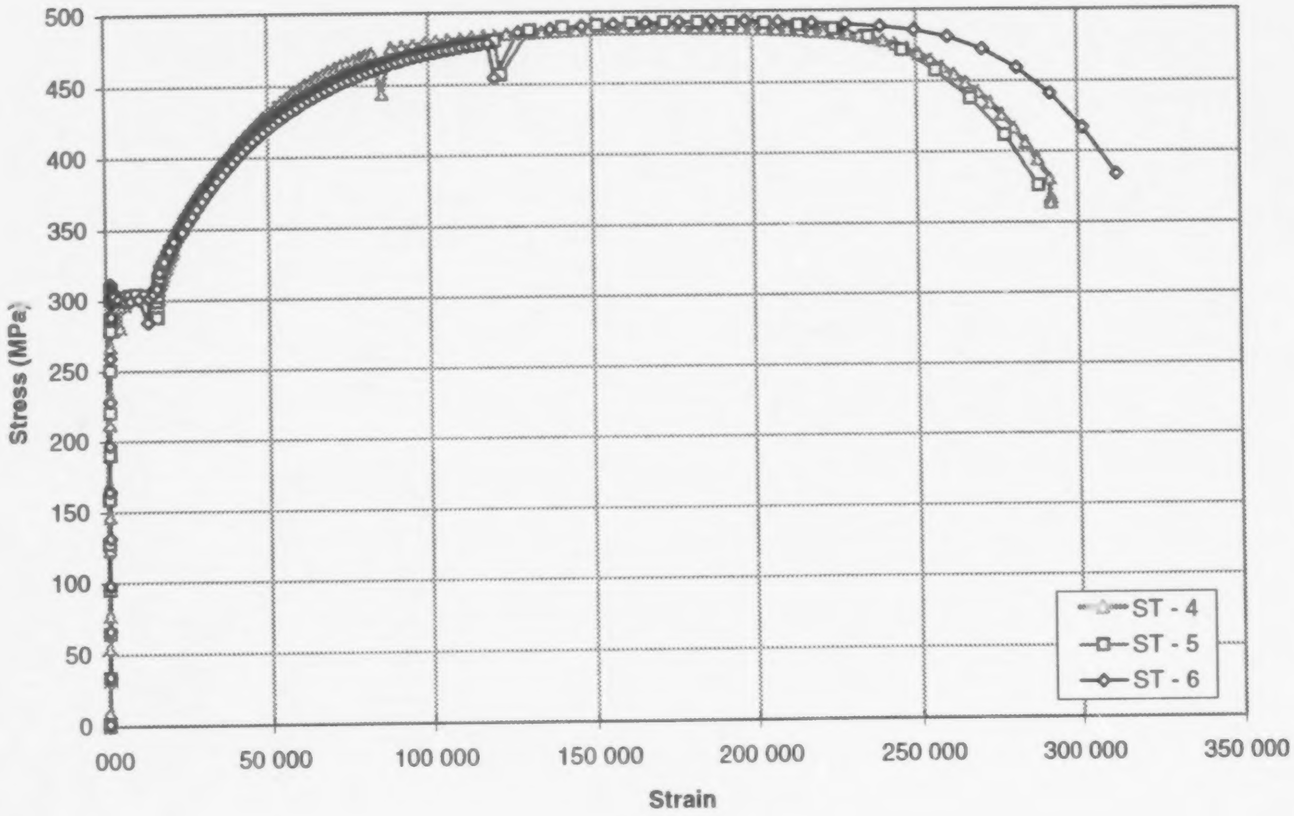

Figure 4.7 Stress versus Strain Plots for Steel Tension Coupons of 19 mm thickness

5. BEHAVIOUR MODEL AND ANALYSIS

5.1 General

The goal of the analysis of the physical test data was to find a suitable means of predicting the load versus deformation behaviour of combination joints. As was mentioned in earlier chapters, this can be done by first analyzing the behaviour of the individual fastening elements contained within a combination joint, and then analyzing the results of the full-scale tests that were conducted. The final prediction model must be based on a rational method of solution so that suitable design recommendations can be devised. The design recommendations must be able to satisfactorily predict joint strengths for both the tests reported herein and tests performed by others.

5.2 Behaviour Model of Individual Fasteners

Using the results of the bolt shear tests and the weld shear tests performed as a part of this research program, a regression analysis of the data was performed and load versus deformation equations were developed. As was outlined in Chapter 4, five single bolts were tested, five transverse weld specimens were tested, and four longitudinal weld specimens were tested. Each set of test results gave individual load versus deformation plots. These plots are shown in Figures 4.2, 4.4 and 4.5. For simplicity of use in the regression analysis, they are plotted only up to the point at which the ultimate load was reached.

The regression analyses were performed using the same analytical expression and the same method used by Fisher [1965]. As demonstrated by Fisher, Eq. [2.1] can be expressed using logarithmic functions. The following equation results:

$$\ln R = \ln R_{ult} + \lambda \ln(1 - e^{-\mu\Delta}) \qquad [5.1]$$

The terms ln R_{ult} and λ can be determined through two simultaneous least squares equations. This is done by varying μ and using the experimental value of R_{ult} as a boundary condition.

A regression analysis was done first for each individual load versus deformation curve from each set of tests, which produces a total of 14 equations. Individual regression analyses were done on each of the sets of data in order to give equal weighting to each test. Using the individual equations derived for each type of fastening element, values of load were found corresponding to regular intervals of deformation. Another regression analysis was then performed on these values, giving a representative load versus deformation equation for each of the individual mechanical fasteners. The following equation for shear resistance as a function of deformation is derived in this way for a representative bolt (3/4 in. diameter, ASTM grade A325) :

$$R = R_{ult}\,(1 - e^{-0.960\Delta})^{0.632} \qquad \text{for } \Delta \le 3.8\,\text{mm} \qquad [5.2]$$

where: R = shear resistance of bolt, as a function of bolt deformation, kN

R_{ult} = ultimate shear resistance of bolt = 349 kN

Δ = bolt shear deformation, mm

The following equations have been derived for the shear resistance versus deformation of representative fillet welds deposited using E48018-1 electrodes

For transverse welds:

$$R = R_{ult}\; l\; d\; (1 - e^{-10.9\times\Delta})^{1.101} \qquad \text{for } \Delta \le 0.52\,\text{mm} \qquad [5.3]$$

For longitudinal welds:

$$R = R_{ult}\; l\; d\; (1 - e^{-7.01\times\Delta})^{0.618} \qquad \text{for } \Delta \le 1.2\,\text{mm} \qquad [5.4]$$

where: R = shear resistance of the weld as a function of shear deformation, kN

R_{ult} = ultimate shear resistance of the weld, kN/mm/mm

= 0.46 kN/mm/mm, for transverse welds

= 0.35 kN/mm/mm, for longitudinal welds

l = weld length, mm

d = nominal weld leg size, mm

Δ = shear deformation of the weld, mm

The results of these predictor equations, along with the experimental data in each case, are shown in Figure 5.1, Figure 5.2, and Figure 5.3 for high-strength bolts, fillet welds loaded transversely, and fillet welds loaded longitudinally, respectively. A comparison of the equations with one another, using the respective transverse and longitudinal weld leg size and length consistent with the full-scale tests, is shown in Figure 5.4.

5.3 Behaviour Model for Combination Joints

5.3.1 General

In order to develop an equation for predicting the load versus deformation behaviour of a full-scale combination joint, the deformations of the individual fasteners present in that joint must be known. It is assumed that the deformations in the component fasteners at any load level are either equal to, or a function of, the average measured total joint deformation. In turn, each of these component deformations corresponds to a certain load resistance for that particular fastener. By summing these load resistances and making an allowance for the resistance in the connection attributed to friction of the plates, an estimate of the load resistance for the entire connection at that particular deformation is obtained.

$$R_{\text{ult joint}} = R_{\text{friction}} + R_{\text{bolts}} + R_{\text{trans}} + R_{\text{long}} \qquad [5.5]$$

where: $R_{ult\ joint}$ = shear resistance of entire connection, as a function of the average connection deformation, kN

$R_{friction}$ = connection resistance contributed from plate friction, kN

R_{bolts} = connection resistance contributed from bolt shear, kN

R_{trans} = connection resistance contributed from transverse weld shear, kN

R_{long} = connection resistance contributed from longitudinal weld shear, kN

5.3.2 Contribution of Frictional Forces

As discussed in Chapter 4, each of the test specimens that had preloaded bolts had a substantially higher ultimate load than did identical specimens without preloaded bolts. Although the frictional forces certainly had a considerable impact on the ultimate load of the combination joint, that effect was not very consistent.

For the design of a slip-critical connection, *The Guide* [Kulak et al., 1987] recommends the following equation:

$$P_{slip} = k_s \times m \times n \times T_i \qquad [5.6]$$

where: P_{slip} = the slip resistance of a bolted joint, kN

k_s = mean slip coefficient

m = number of faying surfaces

n = number of bolts

T_i = initial clamping force of one bolt, kN

In the *Guide,* the authors report that the tension in preloaded high-strength bolts is about 80 % of the ultimate tensile strength of the fastener material. For the purposes of the examination herein, the clamping force per bolt will be taken as 80 % of the bolt tensile strength found in the ancillary experimental program. Thus, this value will be taken as 0.80×218 kN, or 174 kN.

The mean slip coefficient of the plates (k_s) was not determined experimentally in this research program. The *Guide* [Kulak et al., 1987] reports research done by others that evaluates the slip coefficient for a number of surface conditions. The average slip coefficient value for a clean mill scale condition, which was the condition used in the full-scale tests, is 0.33. Using Eq. [5.6], and the parameters of the full-scale tests, the resultant slip load is, $P_{slip} = 459$ kN.

It is important to recognize that Eq. [5.6] is for determining the strength of a bolted slip-critical joint, and it is intended for determining the strength at service loads only. Because the specimens tested in this research program were combination joints and were tested to ultimate, the usefulness of this equation is limited. When a bolted, slip-critical connection starts to slip, it continues to slip until the bolts come into bearing. However, for a combination joint with bolts and welds, the slippage of the joint is partially controlled by the welds; the welds in the connection restrain it from slipping, only allowing the plates to move the amount that the welds are deforming. This is a reasonable explanation as to why there was such an unpredictable increase in strength at ultimate load levels. The increase in ultimate strength ranged from 118 kN to 509 kN.

Still, this increase is not a good indication of how much load the friction in the plates actually contributed. The amount of resistance provided by slip cannot be accurately predicted under these circumstances. In an attempt to establish the friction component that was present in these tests, the load versus deformation curves of each of the 24 full-scale tests were first modeled using a regression analysis. The method and equation used for the analysis were the same as those outlined in Section 5.2. A total of twelve equations for load versus deformation were developed, representing an average of each of the twelve different types of tests. To find the average friction contribution within a particular specimen configuration, the equation representing a specimen with snug-tightened bolts was subtracted from the equation representing a specimen of the same configuration but with preloaded bolts. For example, to find the curve representing the friction contribution for the case of specimens in a negative bearing condition at the start of loading and containing only longitudinal welds, the equation representing the NSL series was subtracted from the equation representing the NPL series.

The six curves found using this procedure were quite dissimilar; they are shown in Figure 5.5, Figure 5.6, and Figure 5.7. The load versus deformation behaviour of the plate friction is significantly affected by the behaviour of the mechanical fasteners in the connection. Thus, it is not surprising the shape of the curves varies so much. The values of maximum load resistance found using these curves, on the other hand, are quite close to the value calculated using Eq. [5.6], as shown in Table 5.1. The average of these values is 434 kN, which corresponds to a difference of only 5 % as compared to the calculated value of Eq. [5.6].

Because the contribution from friction at ultimate is so dependent on the joint configuration and the bearing condition of the bolts in the joint, it is not likely that it can be predicted to an acceptable degree of accuracy. Although the slip resistance of a joint containing only bolts can be reasonably predicted, the slip resistance of a combination joint is quite variable. In each of the cases studied in this test program, the maximum load resistance contributed by the plate friction occurred at a relatively small joint deformation. The load resistance then leveled off to a particular value, which is reflected in the increase in ultimate strength. Perhaps with further testing, a minimum value could be assigned to plate friction contribution. Using the limited data of the testing program reported herein, this minimum value is 118 kN.

5.3.3 Individual Fastener Contribution

5.3.3.1 General

A key issue is how the deformations of the individual fasteners relate to the average connection deformation of the combination joint. At any particular individual fastener deformation, each fastener will contribute a certain amount of that ultimate load resistance, as reflected by Equations [5.2], [5.3], and [5.4]. The three types of mechanical fasteners studied in this testing program rank in ductility (lowest to highest), in the following order: transverse welds, longitudinal welds, and high-strength bolts. The least ductile fastener in a connection will govern the deformation at which the initial failure occurs. Thus, each combination joint should be evaluated using the deformation of this component fastener as a benchmark.

5.3.3.2 Weld Deformation

It is reasonable to assume that all welds, regardless of their orientation (longitudinal or transverse), will begin to contribute resistance at the start of loading. Thus, assuming there is uniform deformation throughout the entire joint, the individual deformations of the welds in the connection should be equal to the average deformation measured in the joint. However, a review of the test data reveals that the joints do not deform uniformly throughout. Rather, variations in deformation exist in the test specimens. Consider, for example, the tests done on specimens with both longitudinal and transverse welds in combination with bolts. The readings from the LVDT's located at the two ends of the transverse welds indicate smaller deformations than those recorded at intermediate locations along the length of the transverse weld (see Figure 5.8). In other words, the transverse welds deform more in certain areas than others. Such local variations were not noticed along the length of the longitudinal welds. However, they are quite prominent in the transverse welds, both for specimens with only transverse welds in combination with bolts and for specimens with transverse and longitudinal welds in combination with bolts. To account for the local deformations in these two groups, a weld deformation profile along the transverse weld was estimated based on the available readings.

For the test specimens with only transverse welds in combination with bolts, the weld deformation profile is modeled using a single parabola. The parabolic equation is determined using the average of the LVDT readings taken at weld failure for each of the eight tests in the group. As shown in Figure 5.8, there are four different LVDTs along the transverse weld on each side of the combination joint. There is an LVDT at each end of the weld and two interior LVDTs, each 85 mm from the end. The average failure deformations recorded by the end LVDTs and the interior LVDTs were 0.26 mm and 0.32 mm, respectively. The parabolic equation determined using these values is as follows.

$$y = (-4.10 \times 10^{-6})\, x^2 + 0.326 \qquad [5.7]$$

where: y = weld deformation, mm

x = distance from the centerline of the joint, mm

The deformation profile produced using this equation is shown in Figure 5.9. The weld deformation at centerline is 0.33 mm, which is also the average maximum deformation attained across the transverse weld at failure. Based on the ancillary tests, it was expected that this value would be 0.52 mm, the failure deformation of transverse welds. The LVDT data indicates that the transverse welds in the full-size tests fractured at a substantially lower deformation than was expected. Presumably, this is because the welds used in the full-scale test specimens were much longer than the welds used in the ancillary tests, thereby having an effect on the deformation characteristics. According to this deformation profile, the average deformation along the transverse weld for this test group is 0.30 mm.

For test specimens with both transverse and longitudinal welds in combination with bolts, a slightly different approach is used to model the deflection profile. In this case, the two ends of the transverse weld are restrained because of the presence of the longitudinal welds on either side. Therefore, the slope of the deformation profile is assumed to be zero at the end points of the transverse weld. Because of the symmetry of the joint, the slope of the profile is also assumed to be zero at the mid point (centerline) of the joint. The deflection profile for this case is then modeled using three parabolas: one parabola representing the interior of the joint, and two symmetrical parabolas representing the outer regions. To create a smooth deflection profile, the slope of the parabolas must be equal at their juncture. The location of this point is one of the unknown factors in determining the profile, as is the deflection at mid-span. Unlike the previous case (transverse welds only), an assumption must be made because there are more equations than unknowns. The assumption made is that the centerline deformation, or maximum deformation, is 0.52 mm. This value is the fracture deformation of transverse welds as found in the ancillary tests. Although this value is not reflected in the results of the second test group, it does produce a smooth curve for these test results, and it is a rational assumption. It is expected that the transverse weld will fail if any portion of the weld is deformed to its fracture deformation. Based on this assumption, the following equations represent the deflection profile at weld fracture for transverse welds in test specimens with both longitudinal and transverse welds. The equations given are for one-half of the joint only; the other half is symmetrical. As was the case for Eq. [5.7], the y-axis denotes the deformation (mm), and the x-axis denotes the distance from the centerline of the joint (mm).

$$y = (3.38 \times 10^{-5})\,(x + 130)^2 + 0.233 \qquad \text{for } x = -130 \text{ to } -68.3 \text{ mm} \qquad [5.8]$$

$$y = (-3.51 \times 10^{-5})\, x^{2} + 0.524 \qquad \text{for } x = -68.3 \text{ to } 0 \text{ mm} \qquad [5.9]$$

The deformation profile produced using these equations is shown in Figure 5.10. The equation reflects a centerline deflection 0.52 mm, which is the value assumed to be the maximum deformation attained across the transverse weld at failure. The average deformation of the transverse weld, according to the assumed deformation profile, is 0.33 mm. For this test group, it was found that the average deformation of the longitudinal welds was 0.23 mm. This coincides with the deformation at the ends of the transverse weld.

The magnitude and effect of the local deformations are likely a function of the size and configuration of the entire combination joint. Each of the different mechanical fasteners present in the combination joint are at different locations and have very different ductilities. This explains why there was such a different deformation profile in the three types of combination joints. The deformation of the entire joint is controlled, at least partly, by the quantity and location of all the mechanical fasteners in the connection.

5.3.3.3 Bolt Deformations

There were no LVDT readings taken at the location of the bolts and therefore it is difficult to assess the amount of shear deformation the bolts undergo at the time of failure of the welds. Also, because the bolts in the connection must first come into bearing before they contribute any shear resistance, they do not necessarily contribute shear resistance at the start of loading. As a result, in addition to any adjustment for local deformation effects, bolt deformation must also be adjusted for bearing condition effects. The distance the plates must deform before the bolts are in bearing, d_s, must be evaluated for each combination joint, then subtracted from the adjusted bolt deformation. At one extreme, the bolts in the connection start out in a negative bearing condition. In this situation, the amount of slip the bolts must undergo is equal to twice the clearance between the bolt and the hole. For this case, it is not expected that the bolts will contribute any resistance at the time of weld failure.

The results of the full-scale tests indicate that there is little variation of the joint deformation in test specimens in the first test group, that is, test specimens with only longitudinal welds in combination with bolts. However, test results from connections with transverse welds in combination with bolts and from the connections with both transverse and longitudinal welds in combination with bolts both indicate that variations in deformation are significant. In both groups, the bolts contribute much less resistance at the time of first weld failure than was expected based on the ancillary tests. In order to determine the shear contribution of the bolts at instant of first weld failure, it is necessary to look at the contribution of the other individual fasteners (the welds) and to then subtract this load from the actual test load. This procedure is outlined in the next section.

5.4 Analysis of Full-Scale Tests

5.4.1 Assumptions

Analysis of the full-scale tests requires that the contribution of each component be assessed. As has already been indicated, the contribution of plate friction and bolt shear is subject to some uncertainty. In analyzing each of the full-scale tests, a number of assumptions are made.

In order to estimate the resistance attributable to the transverse welds, Eq. [5.3] is used. The deformation along the length of the transverse weld cannot be assumed to be uniform, and the deflection profiles developed in Section 5.3.3.2 are used to determine the average deformation for the weld. For the second test group (specimens with transverse welds in combination with bolts), the average deformation of the transverse weld used is 0.30 mm. For the third group (specimens with both longitudinal and transverse welds in combination with bolts), an average deformation of 0.33 mm is used. These values are slightly different likely because the longitudinal welds present in the specimens of group three have an effect on the variations in weld deformation along the length of the transverse weld. Specimens of the first test group did not use transverse welds.

For the purpose of estimating the contribution of load resistance from the longitudinal welds, Eq. [5.4] is used, and the assumption is that the deformation is uniform along the entire length of the weld. For the first test group (longitudinal welds in combination with bolts), the maximum deformation determined in the ancillary tests of longitudinal welds, i.e., 1.2 mm, is used. For the third test group (both longitudinal and transverse welds in combination with bolts), the measured average deformation is used (0.23 mm). In the first test group, the longitudinal welds control the failure deformation; whereas in the third test group the transverse welds control the failure deformation. Thus, it is acceptable that these two values are significantly different from each other. (There were no longitudinal welds used in the second test group specimens.)

For all test specimens with bolts in a negative bearing condition (designated as "N"), the contribution of bolt shear is assumed to be zero. As mentioned earlier, this is a reasonable assumption because the amount of plate slip required to bring the bolts into bearing is greater than the fracture deformation for either longitudinal and transverse welds.

Test specimens with bolts in a positive bearing condition (designated as "P") were analyzed differently. For the first test group (tests with longitudinal welds in combination with bolts) the amount of load resistance attributed to the shearing of the bolts was found using Eq. [5.2]. The bolt shear deformation used in this equation was taken as equal to the deformation of the longitudinal welds, i.e., 1.2 mm. Because the joint deformation in the second and third test groups is not uniform throughout, this method of predicting the bolt deformation will not be satisfactory for these cases. The contribution from bolts can be estimated by analyzing the third set of tests in each of the two groups (PST and PSA), in which the bolts were in a positive bearing condition but here there was no preloading. Thus, because there is little or no friction in the plates for this case, any load resistance other than weld shear can be assumed to be the result of bolt shear. Hence, the contribution from bolt shear is taken as the actual load resistance of the connection minus the calculated weld contribution. It should be noted that using this procedure means that the difference between the theoretical and the actual load resistance is necessarily zero for these particular test results. The average of the two bolt shear resistances calculated for the third set of tests is used subsequently in the fourth set of tests of each group (PPT and PPA).

For test specimens with bolts only tightened to a snug-tight condition (designated as "S"), the amount of load attributed to friction is zero. This assumption is not entirely true as there is likely some friction existing in the plates even if the bolts are not fully preloaded. The assumption is reasonable, however, because the friction resistance contribution to the load resistance of the connection at weld failure will not be very large as compared to the total load resistance of the connection.

For test specimens with preloaded bolts (designated as "P"), it is assumed that the amount of load resistance attributed to friction is equal to the estimated values of friction contribution determined in Sections 4.2.2 to 4.2.4.

5.4.2 Summary of Results

It was anticipated originally that the contribution of each of the mechanical fasteners could be estimated using Equations [5.2], [5.3], [5.4], and [5.6]. Analyses of the test results, however, showed that this is not necessarily true. The effect of the frictional component to the total load, for instance, is very unpredictable. Bolt shear resistance can be reasonably estimated in combination joints that have only longitudinal welds; however, when transverse welds are present the bolt shear contribution is significantly lower than expected. Furthermore, although the welds in the connections behaved generally as expected, local variations in the deformations are very prominent. This has the effect of decreasing the failure deformations in some cases.

Using the assumptions outlined in the previous section, an estimate of the contribution of each individual fastener can be developed for each full-scale test. The results of the analysis for the first, second, and third test groups are shown in Table 5.2, Table 5.3, and Table 5.4, respectively. Shown in the tables are the calculated loads, the actual load obtained in the test, and the ratio of calculated to actual load. The total calculated load is the sum of the calculated weld load, the calculated bolt shear load, and the estimated friction load. The test load is the experimental load found in the test program. A summary of the analysis of the results from each of the three test groups follows.

The analysis results of the first test group (longitudinal welds in combination with bolts) are shown in Table 5.2. The experimental longitudinal weld fracture deformation (1.2 mm) was used in Equations [5.2] and [5.4] to determine the load contribution of the bolts and the longitudinal welds, respectively. The friction loads used (for NPL and PPL), are the average estimated values, as cited in Section 4.2.2. Overall, for this test group the ratio of calculated load to test load is an average value of 0.98, indicating a good prediction of the connection strength.

The analysis results of the second test group (transverse welds in combination with bolts) are shown in Table 5.3. The load contribution of the transverse welds was calculated employing Eq. [5.3] and using the estimated average of the deflection profile for this test group, which is 0.30 mm. The contribution from bolt shear for the PST specimens was determined by subtracting the calculated weld load from the actual test load. The average value of this calculated bolt shear is subsequently used for the PPT specimens. The friction contribution (for NPT and PPT) is assumed to be the average estimated values, as cited in Section 4.2.3. For the NST, NPT, and PPT specimens, the average ratio of calculated load to test load was 1.02. This estimate of the joint strength is within an acceptable range. For PST test series, the calculated load was determined using the actual test load. Thus, the ratio of calculated load to test load is necessarily equal to 1.0 for this case.

The results of the analysis of the third test group (specimens with both longitudinal and transverse welds in combination with bolts) are shown in Table 5.4. Again, the transverse weld contribution is calculated using Eq. [5.3], this time with a deformation of 0.33 mm (average estimated deformation for this test group). The longitudinal weld contribution is calculated according to Eq. [5.4] and using a deformation value of 0.23 mm. For the PSA series, the bolt load is calculated by subtracting the calculated weld load from the actual test load. The average of this calculated bolt load is subsequently used in the PPA series. The contribution from friction, applicable to the NPA and PPA series, is taken as the average estimated values, as cited in Section 4.2.4. The average ratio of calculated load to test load for the NSA, NPA, and PPA specimens is 1.09. This value is relatively high, and it indicates an unconservative prediction. The inaccuracy of this prediction is likely because of the fact that there are many assumptions made. For the PSA specimens, the calculated load was determined using the actual test load, thus, the ratio of calculated load to test load is necessarily equal to 1.0.

5.4.3 Discussion

In reviewing the results of the test data analysis, at least two conclusions can be drawn. First, it has already been stated that the friction resistance at ultimate load is very unpredictable. In some of the tests, the contribution attributed to the preloading in the bolts is quite substantial, whereas in other tests this contribution is very small. It seems reasonable, therefore, to use a lower bound solution when taking friction into account.

The second important observation is that the bolts do not contribute much shear resistance when they are used in combination with transverse welds. In the first test group (no transverse welds), the contribution of load resistance attributed to the bolts was very close to what was expected. However, for the second and third test groups (both use transverse welds) the bolts contributed substantially less load resistance than was anticipated, even for those situations where the bolts were put into a positive bearing condition before the start of loading. Even in the most favourable conditions of this test program, the bolts only contributed about 6 % of their ultimate shear resistance when they were used in conjunction with transverse welds.

Table 5.1 Values of Maximum Load from Plate Friction Curves

Series Designation*	Maximum Load kN	Deformation at Maximum Load mm
NL	436	1.550
PL	439	0.170
NT	496	0.065
PT	549	0.470
NA	322	0.085
PA	364	0.550

* N = negative bearing condition
P = positive bearing condition
L = longitudinal welds only
T = transverse welds only
A = both longitudinal and transverse welds

Table 5.2 Full-Scale Tests: Longitudinal Welds plus Bolts

Test No.	Weld Leg Size mm	Calculated Loads				Test Load kN	Ratio of Calculated to Test Load
		Weld Load kN	Friction Load kN	Bolt Load kN	Total Load kN		
NSL-1	6.28	1224	0	0	1224	1308	0.94
NSL-2	5.88	1145	0	0	1145	1234	0.93
NPL-1	6.28	1224	470	0	1694	1776	0.95
NPL-2	6.16	1199	470	0	1669	1706	0.98
PSL-1	6.30	1228	0	1098	2326	2233	1.04
PSL-2	6.36	1238	0	1098	2336	2378	0.98
PPL-1	6.19	1206	118	1098	2422	2418	1.00
PPL-2	6.45	1257	118	1098	2473	2428	1.02

Longitudinal weld and bolt deformation = 1.2 mm.

Table 5.3 Full-Scale Tests: Transverse Welds plus Bolts

Test No.	Weld Leg Size mm	Calculated Loads				Test Load kN	Ratio of Calculated to Test Load
		Weld Load kN	Friction Load kN	Bolt Load kN	Total Load kN		
NST-1	6.37	1453	0	0	1453	1380	1.05
NST-2	6.29	1435	0	0	1435	1400	1.03
NPT-1	6.06	1383	291	0	1674	1676	1.00
NPT-2	6.05	1381	291	0	1672	1685	0.99
PST-1	6.08	1386	0	88	1474	1474	1.00
PST-2	6.51	1486	0	98	1584	1584	1.00
PPT-1	6.48	1478	509	93	2080	2111	0.99
PPT-2	6.67	1522	509	93	2124	1965	1.08

Transverse weld deformation = 0.30 mm.

Values in shaded areas must necessarily be 1.00 : see Section 5.4.1

Table 5.4 Full-Scale Tests: All-Around Welds plus Bolts

Test No.	Weld Leg Size, mm		Calculated Loads				Test Load kN	Ratio of Calculated to Test Loads
	Trans. Weld	Long. Weld	Weld Load kN	Friction Load, kN	Bolt Load kN	Total Load kN		
NSA-1	5.78	6.55	2448	0	0	2448	2231	1.10
NSA-2	5.75	6.26	2391	0	0	2391	2195	1.09
NPA-1	5.90	6.68	2497	196	0	2693	2390	1.13
NPA-2	5.93	6.56	2484	196	0	2680	2428	1.10
PSA-1	6.01	6.39	2473	0	85	2558	2558	1.00
PSA-2	5.85	6.28	2418	0	40	2458	2458	1.00
PPA-1	6.18	6.49	2530	372	63	2965	2894	1.02
PPA-2	6.20	6.67	2565	372	63	3000	2692	1.11

Transverse deformation = 0.33 mm; longitudinal deformation = 0.23 mm.

Values in shaded areas must necessarily be 1.00 : see Section 5.4.1

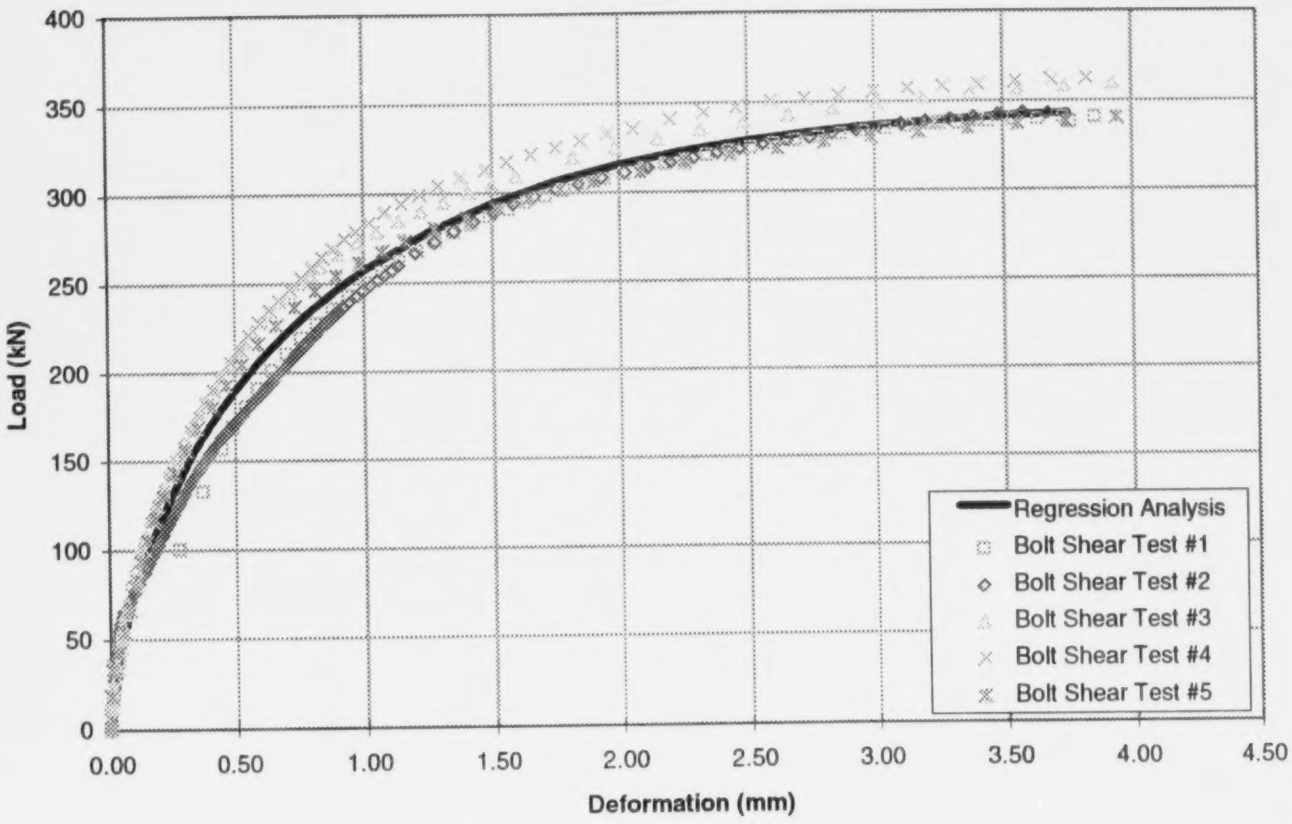

Figure 5.1 Load versus Deformation of Bolts in Shear

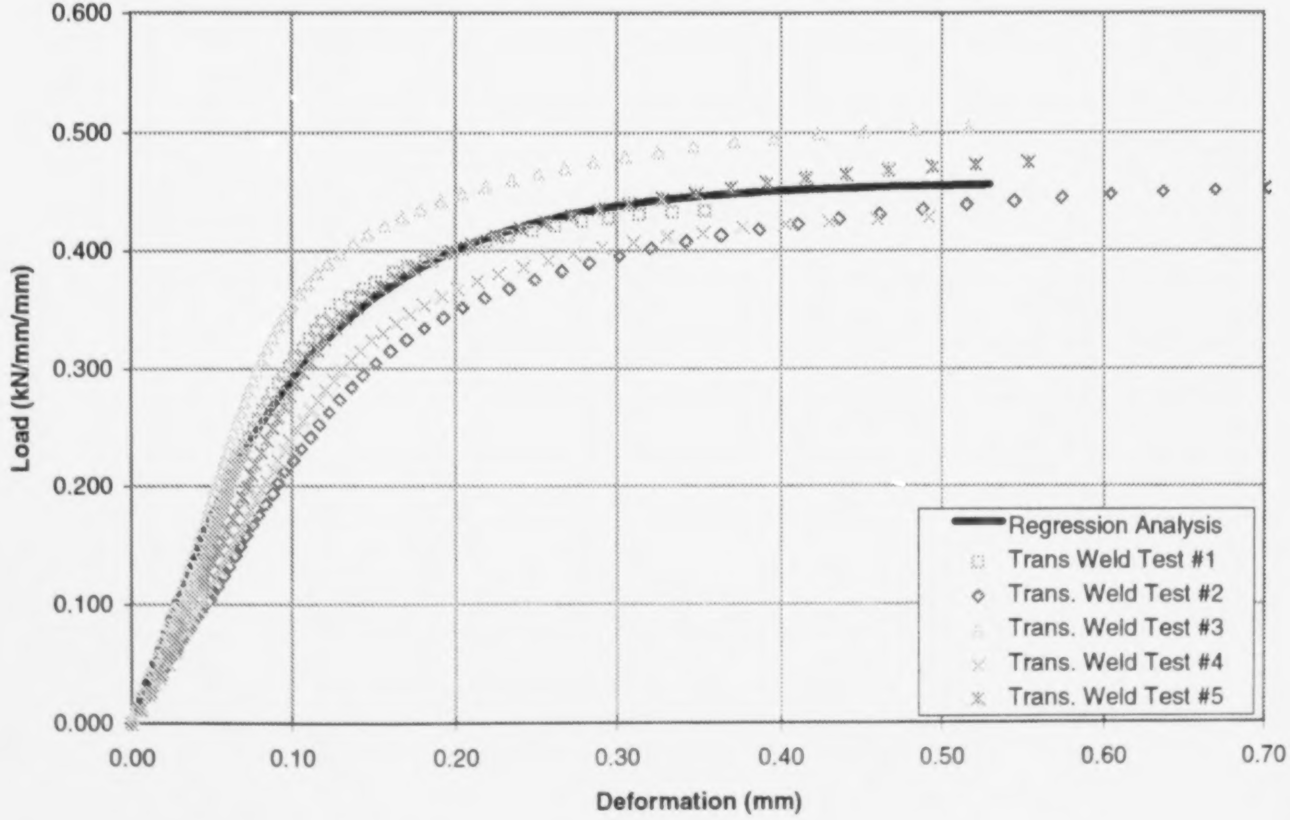

Figure 5.2 Load versus Deformation of Transverse Welds in Shear

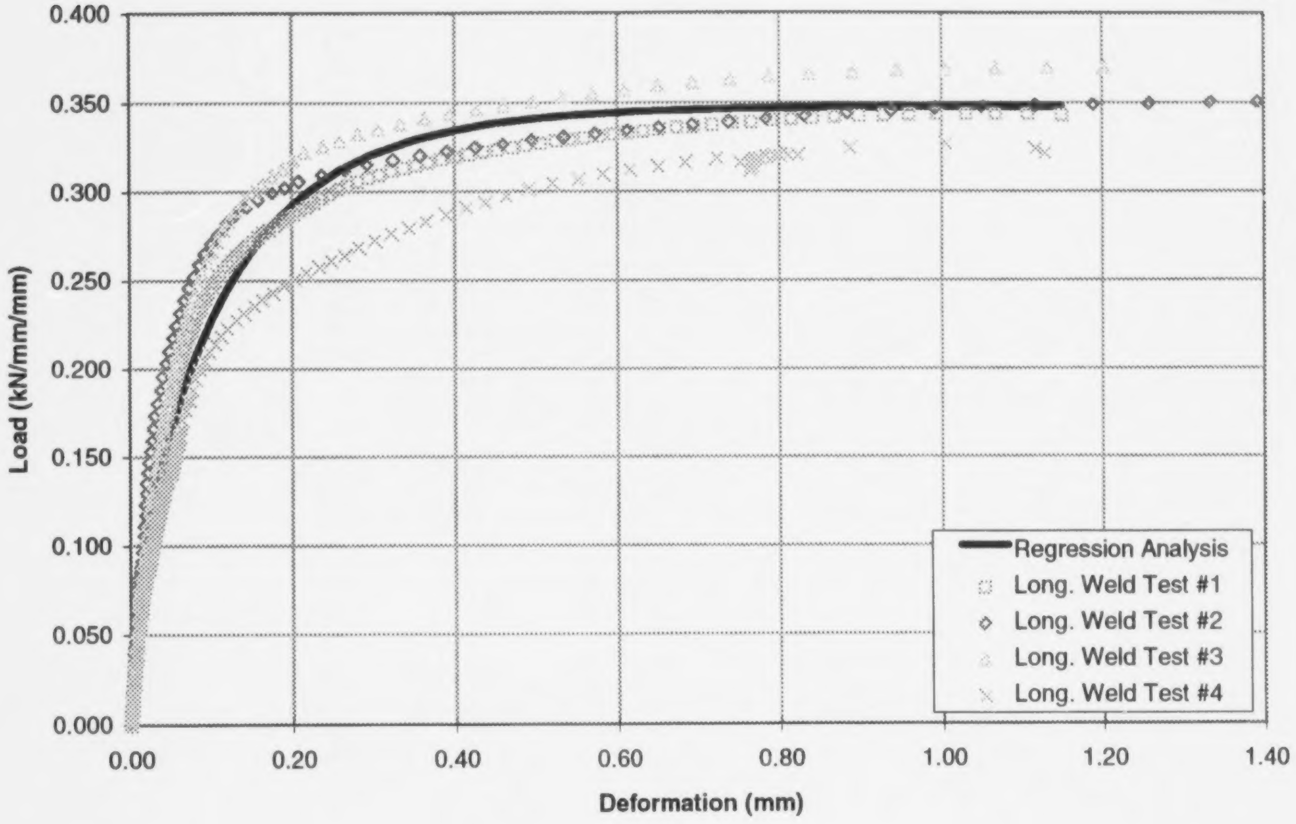

Figure 5.3 Load versus Deformation of Longitudinal Welds in Shear

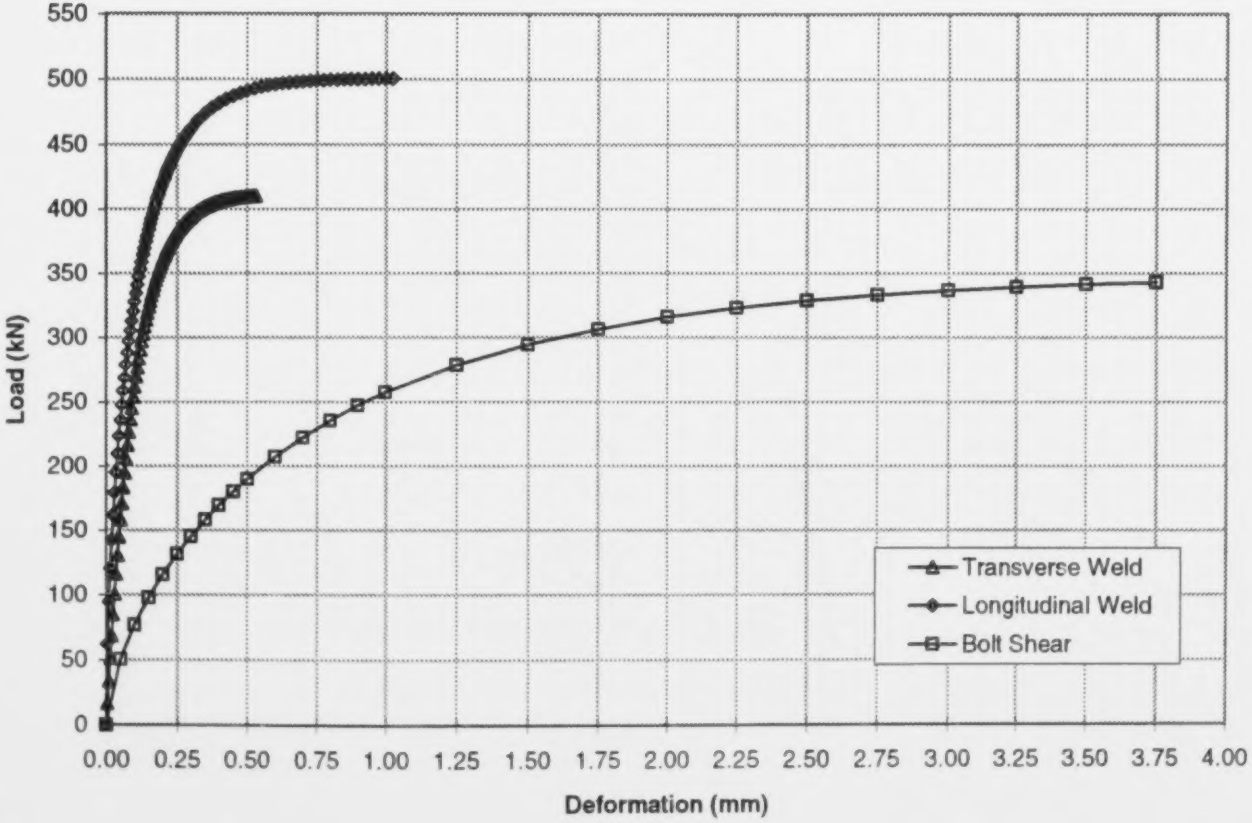

Figure 5.4 Load versus Deformation of Individual Fasteners

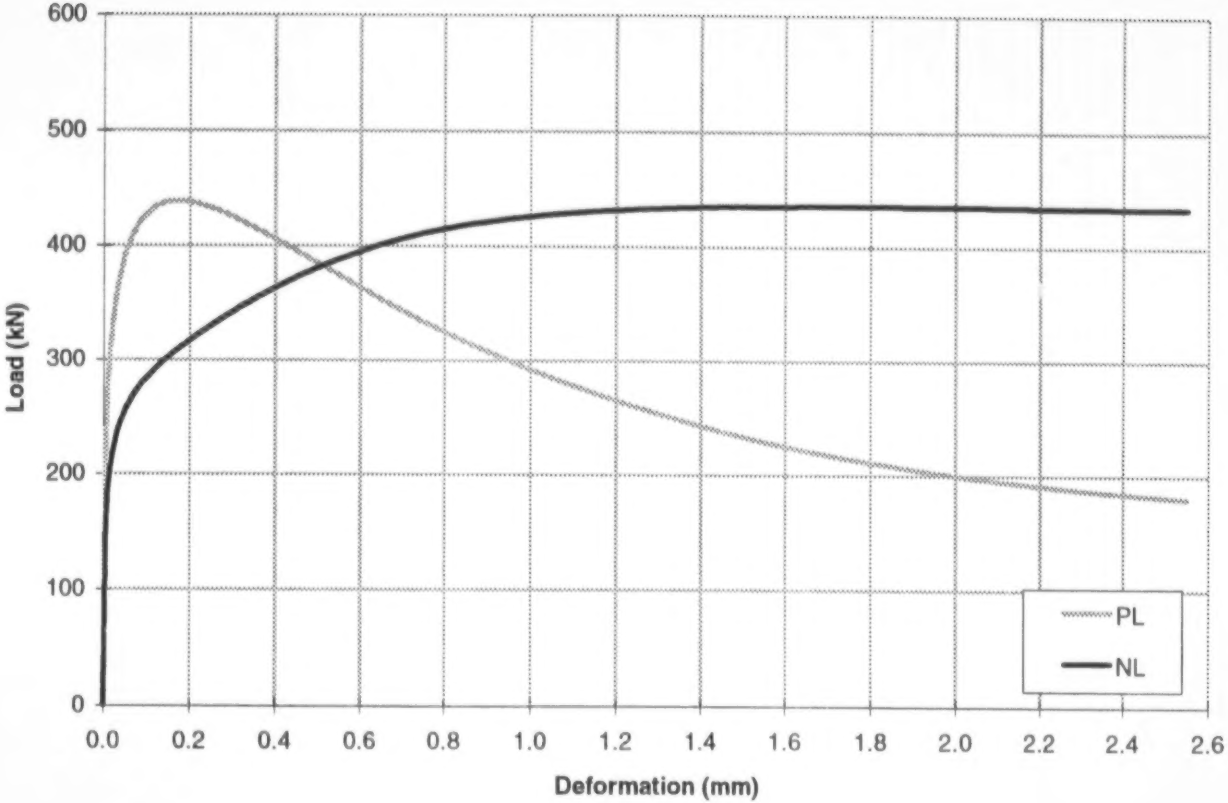

Figure 5.5 Load versus Deformation Curves for Plate Friction in Specimens with Longitudinal Welds

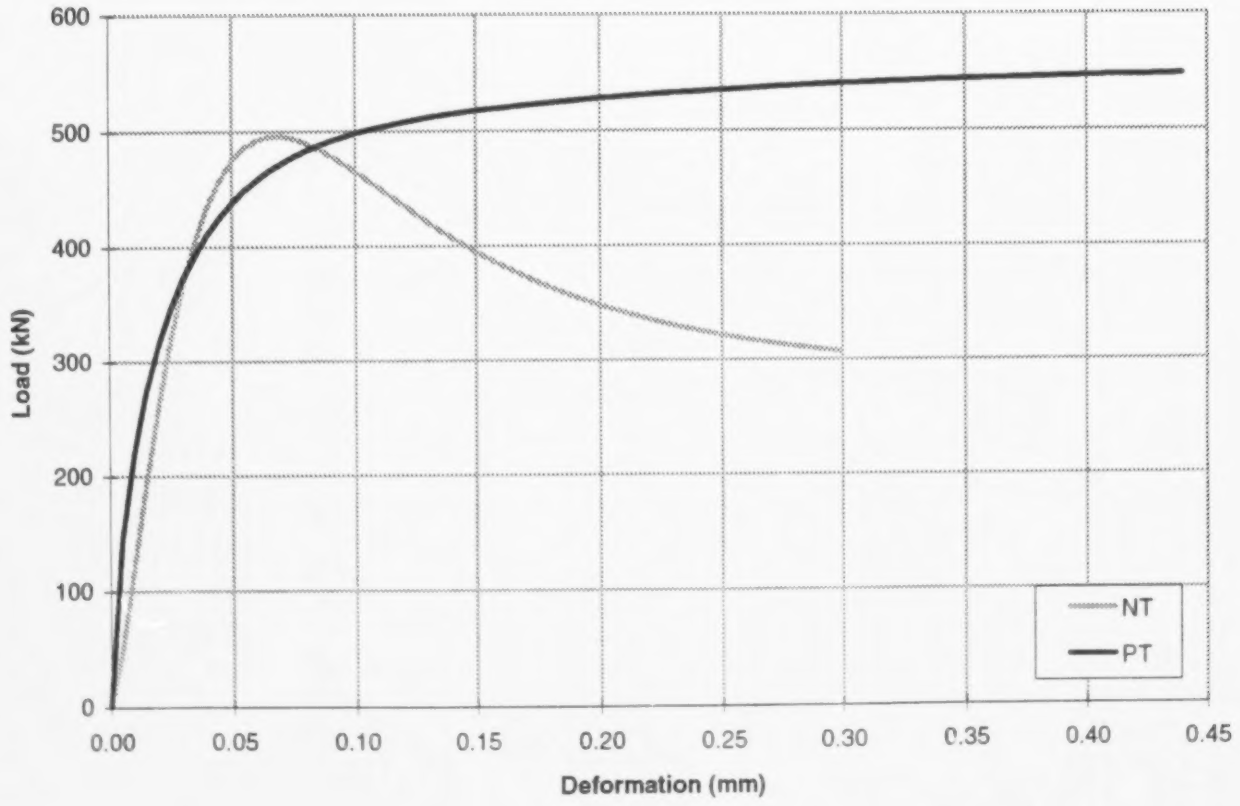

Figure 5.6 Load versus Deformation Curves for Plate Friction in Specimens with Transverse Welds

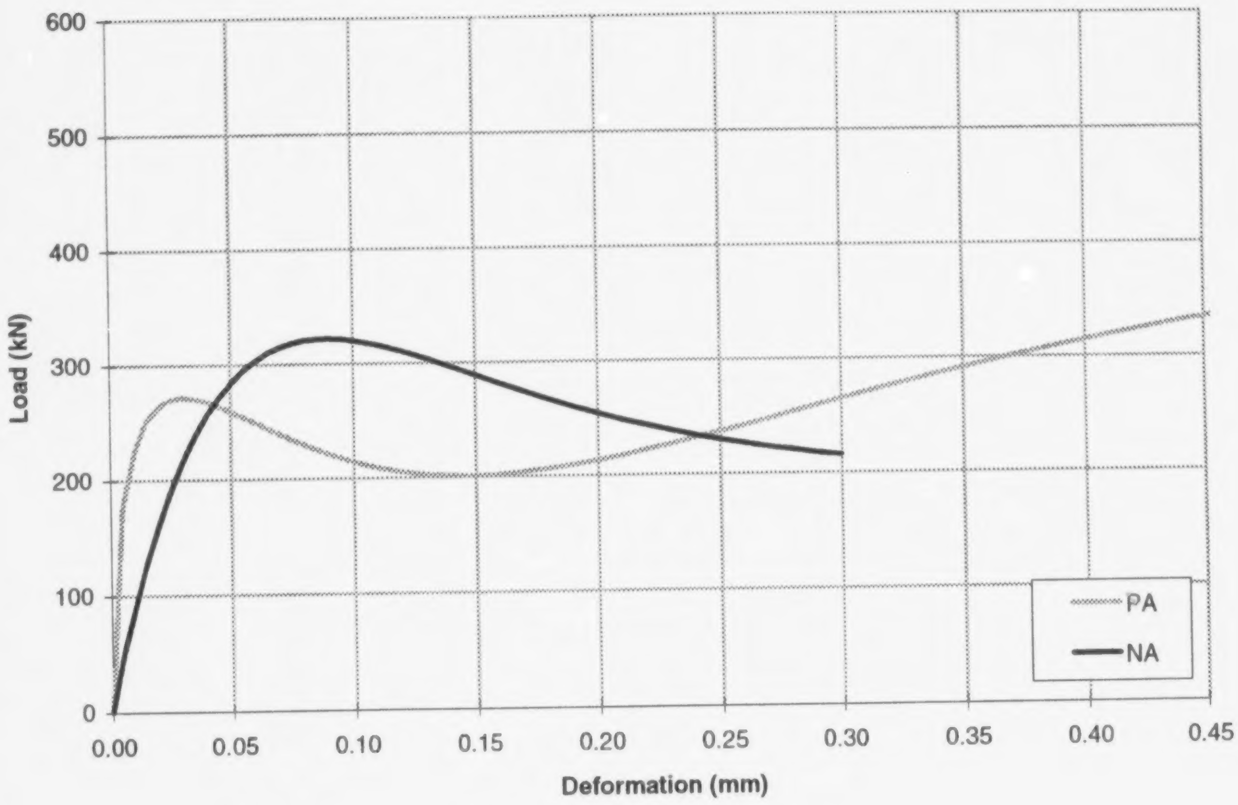

Figure 5.7 Load versus Deformation Curves for Plate Friction in Specimens with both Types of Welds

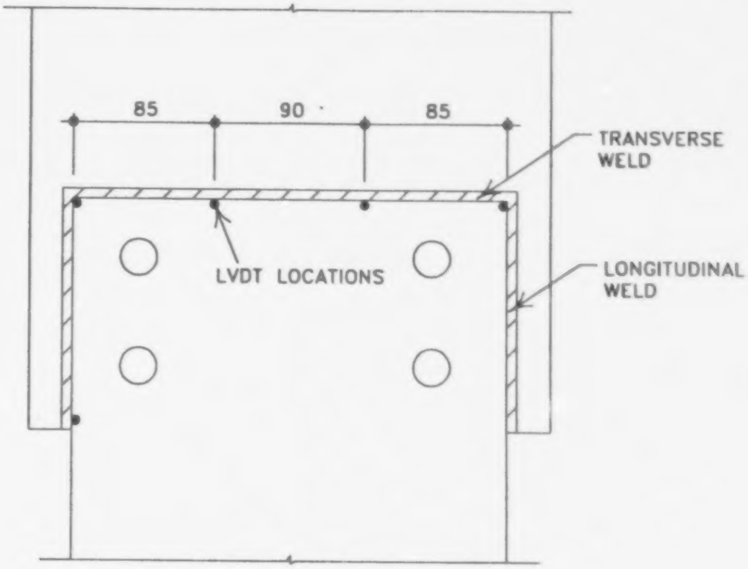

Figure 5.8 Location of LVDTs

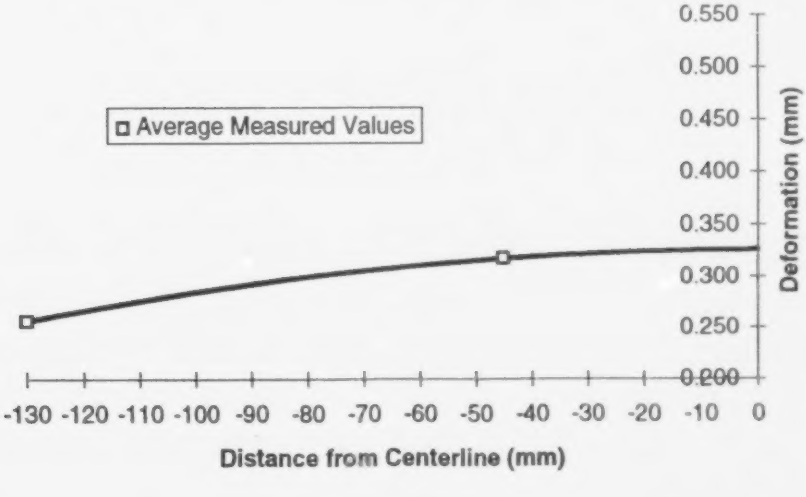

Figure 5.9 Deformation Profile of Transverse Weld (Group 2)

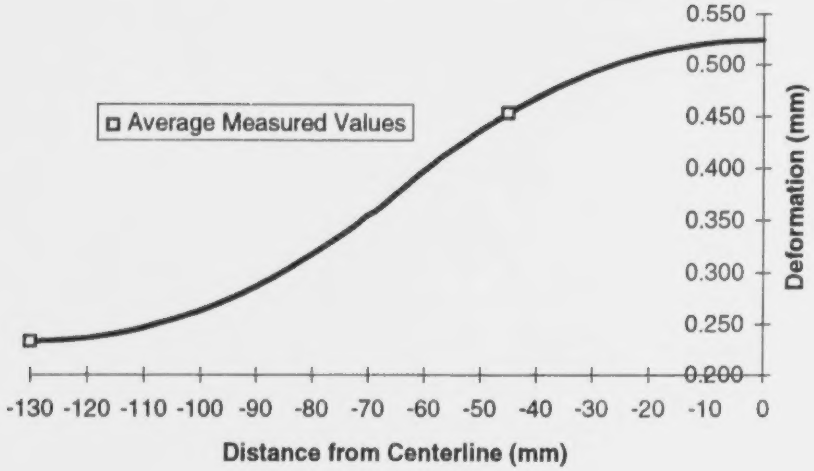

Figure 5.10 Deformation Profile of Transverse Weld (Group 3)

6. DESIGN RECOMMENDATIONS

6.1 General

In the previous chapter, an analysis was carried out that enabled a comparison to be made with the results of the experimental program reported herein. Estimates were made of the individual fastener contributions to the total shear strength of the combination joints tested. Based on the reliability of these estimates, design recommendations can now be made. The recommendations developed herein are for the design of tension connections that use both high-strength bolts and fillet weld acting in the same shear plane. The equations developed in Section 5.2 are not used in the design procedure because their application would be too complicated for practical applications. Rather, the results obtained using the equations are simplified into procedures more suitable for design purposes.

Because the scope of the testing program was necessarily limited, the recommendations may not be as useful for situations where the parameters are significantly different from those used in the test program. For example, the full-scale test specimens of the experimental program reported herein all have four 3/4 in. diameter bolts of ASTM grade A325. The design recommendations may not render as good a prediction of shear strength for a combination joint with many bolts, or a joint with bolts of a different grade or of significantly different size. Also, the fillet welds used in the experimental program all had a nominal leg size of 6 mm, and were deposited using E48018-1 weld electrodes. Again, weld leg size and weld electrode grade presumably have an effect on the ductility characteristics of an entire combination joint. As the parameters of the combination joint change, the design recommendations will become less applicable. To develop an extensive design procedure, more research in the area of combination joints is needed. Nevertheless, in the test program reported herein the bolt sizes and grade and weld sizes and electrode used are all representative of reasonable fabrication practice and procedures.

6.2 Recommendations for Combination Joints

6.2.1 Contribution of Frictional Forces

Although the level of the plate friction resistance at ultimate load was found to be quite variable, it is difficult to discount its effects completely. Specimens with preloaded bolts were always found to be stronger, at least to some degree, than identical specimens with bolts tightened only to the snug-tight condition. As estimated in Section 4.2, this strength increase ranged from 118 kN to 509 kN. The lower of these two values, 118 kN, is 26 % of 459 kN, the estimated value of the slip resistance of the joint calculated using Eq. [5.6]. In order to account for the effects of slip resistance at ultimate load in a combination joint (using a lower bound solution) this is rounded down to 25 %. This value is simply an estimate of the least amount of contribution from friction at ultimate load, based on the limited results of the testing program presented herein. Thus, the equation for determining the connection resistance contributed to plate friction is as follows.

$$R_{\text{friction}} = 0.25 \times P_{\text{slip}} \qquad [6.1]$$

where: P_{slip} = the slip resistance of a bolted joint, kN

6.2.2 Contribution from Transverse Weld Shear

When transverse welds are used in a combination joint, the shear resistance attributable to them is assumed to be their ultimate shear strength. The assumed weld deformation profiles developed using the LVDT measurements of the testing program indicated that rather than deforming 0.52 mm (the expected transverse weld fracture deformation), the transverse welds in the second test group deformed 0.30 mm on average, and the transverse welds in the third test group deformed only 0.33 mm on average. Thus, the transverse welds did not deform as much as expected from the ancillary tests. Nevertheless, they contribute nearly all of their ultimate shear resistance. For example, using $\Delta = 0.30$ mm in Eq. [5.3] shows that at this deformation the resistance of the transverse weld is 96 % of its ultimate shear resistance. For simplicity in design, therefore, this value can be taken as 100 % of the ultimate shear resistance. Thus for a given weld length, l, and given weld leg size, d, the contribution of the transverse weld component is as follows:

$$R_{trans} = R_{ult\,trans} \times l \times d \qquad [6.2]$$

where: $R_{ult\,trans}$ = ultimate shear resistance of the transverse weld, kN/mm/mm

6.2.3 Contribution from Longitudinal Weld Shear

The amount of connection resistance contributed from longitudinal weld shear depends largely upon whether or not transverse welds are also used in the connection. If only longitudinal welds are used, then the connection resistance from longitudinal weld shear is assumed to be the longitudinal weld ultimate shear strength. In the absence of transverse welds, the total connection resistance of the combination joint is evaluated when the longitudinal welds fail; hence, the assumption is valid. This assumption is also supported by test results of the experimental program.

If transverse welds are also present in the connection, then the contribution of the longitudinal welds must be evaluated at the time the transverse welds reach their fracture deformation since they are the least ductile type of fastener. In this case, the longitudinal weld deformation is also somewhat affected by the variation in deformation along the transverse weld. Measurements from the third test group of the experimental program show that, on average, the longitudinal welds deform only 0.23 mm at the time of transverse weld failure. Substituting this deformation into Eq. [5.3] shows that the longitudinal welds contribute 87 % of their ultimate shear capacity when used in combination with transverse welds and high-strength bolts. This value is likely to change slightly for different joint configurations. For simplicity in design, the value is taken as 85 %.

The equations for connection resistance contributed by longitudinal weld shear for a given weld length, l, and a given weld leg size, d, are as follows:

For combination joints with only longitudinal welds—

$$R_{\text{long}} = R_{\text{ult long}} \times l \times d \qquad [6.3]$$

For combination joints with both longitudinal and transverse welds—

$$R_{\text{long}} = 0.85 \times R_{\text{ult long}} \times l \times d \qquad [6.4]$$

6.2.4 Contribution from Bolt Shear

As was the situation for longitudinal welds, the amount of connection resistance contributed from bolt shear also depends largely upon whether or not transverse welds are present in the connection. In addition, this resistance will also depend on the bearing condition of the bolts at the start of loading.

In the experimental program, it was found that the shearing of the bolts contributed very little load to the connection resistance when transverse welds were used in the connection. Even when the bolts were in a positive bearing condition at the start of loading, their estimated contribution to the total resistance at the time of weld failure was less than 100 kN. The positive bearing condition is the so called "best case" for the bolts, yet the bolts only contribute about 6 % of their ultimate load resistance. Even in the most favourable conditions for combination joints with transverse welds, the contribution of shear resistance from the bolts is barely noticeable. Therefore, for design purposes it is assumed that bolts do not contribute any shear resistance to the connection when used in combination with transverse welds.

Connection resistance contributed from bolt shear is available when the bolts are used in combination with longitudinal welds. However, in order to evaluate the resistance contributed by bolt shear to the connection, the amount of slip the plates must move to put the bolts into bearing (d_s) must first be evaluated. If the plates must slip a distance greater than 1.2 mm (the fracture deformation of longitudinal welds), then the bolts will not contribute any resistance in combination with the welds. Under experimental conditions, the condition of positive bearing can be introduced. For this situation, it was found that the bolts contribute about 79 % of their ultimate shear strength. Conservatively, this number will be rounded down to 75 %. In practice, however, it is unlikely that the bolts will be in such a favourable bearing condition as that created in the laboratory. A reasonable assumption is to contemplate that a connection considered to be in positive bearing is more likely to have its bolts situated half-way between centered on the hole and actual positive bearing. For a standard bolt hole, drilled 1.6 mm (1/16 in.) larger than the nominal bolt diameter, the total slip distance possible is equal to 3.2 mm. Thus, using this assumption, the plate slippage required before the bolts are in full bearing (d_s) is 0.8 mm. Using Eq. [5.2], and the estimated fracture deformation of longitudinal welds (1.2 mm), it is found that in this situation the bolts contribute about 49 % of their ultimate shear strength. For simplicity, this number is rounded up to 50 %.

The equations for connection resistance contributed by bolt shear for n number of bolts are as follows:

For combination joints with transverse welds and for combination joints with bolts not in a positive bearing condition—

$$R_{\text{bolts}} = 0 \qquad [6.5]$$

For combination joints with only longitudinal welds and bolts in a positive bearing condition (test conditions)—

$$R_{\text{bolts}} = 0.75 \times n \times R_{\text{ult bolts}} \qquad [6.6]$$

For combination joints with only longitudinal welds and bolts in positive bearing condition (field conditions)—

$$R_{\text{bolts}} = 0.50 \times n \times R_{\text{ult bolts}} \qquad [6.7]$$

Finally, to determine the shear resistance of the entire joint, $R_{\text{ult joint}}$, the values of R_{frict}, R_{long}, R_{trans}, and R_{bolts}, as determined according the preceding recommendations are substituted into Eq. [5.5].

6.3 Design Criteria

The design recommendations presented herein are intended for prediction of the ultimate load resistance of the connections. When applying these recommendations to a limit states design, the loads and resistances should be factored accordingly. In examining the results of the full-scale tests, only the load resistance at first weld failure has been considered thus far. However, as mentioned in earlier chapters, the ultimate load does not necessarily correspond to the load at first weld failure. Since the objective of the design procedure is to predict the ultimate load, each of the different failure mechanisms must be considered. Accordingly, the ultimate strength of a combination joint is the greatest of:

1) The ultimate strength of the bolts only
2) The ultimate strength of the welds only
3) The combined strength of the longitudinal welds and the high-strength bolts, as calculated using the provisions of Section 6.2
4) The combined strength of all the welds with the high-strength bolts, as calculated using the provisions of Section 6.2.

6.4 Evaluation of Design Recommendations

6.4.1 General

In order to check the validity of the design recommendations presented herein, a comparison is made between the joint shear strength calculated using the recommendations and actual experimental values of joint shear strength. Three independent sets of experimental results are examined—results from the research reported herein, results from the research of Holtz and Kulak [1970], and results from the research of Jarosch and Bowman [1985]. Each research program involves tests of combination joints with different configurations.

6.4.2 Research Reported Herein

The comparison of the joint shear strength calculated using the recommendations with the actual measured joint shear strength from the research reported herein is shown in Table 6.1. The bolts used in these tests were 3/4 in. diameter ASTM grade A325. Welds were deposited using an E48018-1 weld electrode. The configuration of each test specimen, with the exception of weld leg size, is listed in Table 3.1. The weld leg sizes are listed in Table 6.1. The ultimate shear strength values of the bolts and welds as found in the ancillary tests, are presented in Tables 4.3 and 4.6, respectively.

To determine the value of R_{trans}, Eq. [6.2] is used. To determine the value of R_{long}, Eq. [6.3] is used for specimens designated with an "L" (longitudinal welds only) and Eq. [6.4] is used for specimens designated with an "A" (both types of welds). Equations [5.6] and [6.1] are used to determine the value of R_{frict}. For specimens with preloaded bolts (designated with a P), this value is 115 kN; for specimens with bolts tightened only to the snug-tight condition, this value is zero. To determine the value of R_{bolts}, Eq. [6.5] is used for specimens designated with a "T" (transverse welds only) or an

"A" (both types of welds), and Eq. [6.6] is used for specimens designated with an "L" (longitudinal welds only).

Using these recommendations, the ratio of calculated load to actual load is close to unity for most cases (see Table 6.1). The average values for each of the three test groups, L, T, and A, were 0.92, 0.95, and 1.03, respectively. Taking all of the results together, the mean value of this ratio of calculated load to actual load is 0.97, standard deviation 0.09. The values predicted by the recommendations are slightly conservative in some cases because the contribution from plate friction is conservative. There are no instances in which the recommendations produce a particularly unconservative prediction. In the worst case, the ratio of calculated load to test load is 1.10.

It should also be noted that for the first four test specimens in Table 6.1, the ultimate bolt shear governed the design. For these specimens, it was predicted that the maximum shear strength of the connection, 1396 kN, would occur at the time of bolt failure (i.e., after the welds had failed). In the first two of these cases, the physical tests corresponded to this prediction, that is, the welds failed first followed by continued loading of the bolts. Maximum load in these cases was reached at the time of bolt failure. In the other cases, however, the maximum load resistance of the joint was reached at the time of weld failure.

6.4.3 Research by Holtz and Kulak [1970]

A comparison of the joint shear capacity obtained in the tests conducted by Holtz and Kulak [1970] with the calculated capacity using the recommendations of Section 6.2 is shown in Table 6.2. The specimens tested by Holtz and Kulak had either longitudinal fillet welds or transverse fillet welds in combination with 3/4 in. diameter bolts of ASTM grade A325. The physical properties of the bolts and welds used by Holtz and Kulak were the same as those used by Crawford and Kulak [1971], and Butler et al. [1972], respectively. These properties are listed in Tables 4.3 and 4.6.

For the first six tests listed in Table 6.2, two bolts were used; only one bolt was used in the last three tests. Bolts in the first three tests were tightened only to a snug-tight condition but the holes were "fitted," that is, the standard 1/16 in. clearance was not provided. The bolts had to be lightly tapped into their holes. For the specimens in the last six tests, the standard hole clearance was provided, and the bolts were preloaded.

Although the bolt tensile strength is not reported, it can be estimated using the value of bolt shear strength. As mentioned in Chapter 4, *the Guide* [Kulak et al., 1987] suggests that the shear strength of a bolt is 0.62 times its ultimate tensile strength. The ultimate bolt shear strength for these tests was 579 MPa. Thus, the corresponding bolt tensile strength is 934 MPa, or 200 kN. Substituting this into Eq. [5.6], the value of P_{slip} is equal to 106 kN per preloaded bolt. The value of R_{frict} is therefore 27 kN per preloaded bolt, as per Eq. [6.1].

The type of weld used for each test and the measured values of weld length and weld leg size are shown in Table 6.2. The values of R_{weld}, as shown in Table 6.2, correspond to the values of R_{trans} and R_{long} for specimens with transverse welds and specimens with longitudinal welds, respectively. For the former case, R_{weld} is calculated using Eq. [6.2]; for the latter case, R_{weld} is calculated using Eq. [6.3].

It was found by Holtz and Kulak [1970], that the use of "fitted" bolts did not simulate a positive bearing condition very well. For this reason, rather than using Eq. [6.6] to find the value of R_{bolts} (for the first three tests in Table 6.2), Eq. [6.7] was used. The value of R_{bolts} for the fourth to sixth test is also calculated using Eq. [6.7]. For the last three tests, R_{bolts} is calculated using Eq. [6.5].

The predictions made using the recommendations of Section 6.2 are reasonably close to the actual values of joint shear strength found by Holtz and Kulak [1970]. On average, the ratio of predicted to test values was 1.01, standard deviation 0.05. The most conservative estimate had a ratio of 0.95, and the most unconservative had a ratio of 1.08. The combined strength of the bolts and welds governed the predicted ultimate joint strength in all cases.

6.4.4 Research by Jarosch and Bowman [1985]

A comparison of the actual joint shear capacity obtained from combination joint tests conducted by Jarosch and Bowman [1985] with predictions made using the recommendations of Section 6.2 is shown in Table 6.3. The bolts used in the Jarosch and Bowman tests were 3/4 in. diameter and were ASTM grade A325. Jarosch and Bowman did not carry out any tests on their individual mechanical fasteners to determine their ultimate shear strength. Rather, they assumed the same values as Holtz and Kulak had determined experimentally [1970]. As mentioned in Section 6.4.3, the physical properties of the bolts and welds used by Holtz and Kulak were the same as those used by Crawford and Kulak [1971] and by Butler et al. [1972], respectively. These properties are listed in Tables 4.3 and 4.6.

Since there are no measured values of weld length or weld leg size in the Jarosch and Bowman report, nominal values have been used for the examination herein. The nominal weld lengths are 279 mm for the transverse case and 457 mm for the longitudinal case. The nominal leg size for all welds is 1/4 in. (6.4 mm). The value of R_{trans} is calculated using Eq. [6.2]; the value of R_{long} is calculated using Eq. [6.3] for the third and fourth specimens (Table 6.3), and Eq. [6.4] is used for the fifth and sixth specimens.

Two preloaded high-strength bolts were used in all of the combination joint tests conducted by Jarosch and Bowman [1985]. Using the method described in Section 6.4.3, the value of R_{frict} is taken as 27 kN per bolt.

The actual bearing condition of the bolts used by Jarosch and Bowman [1985] is not known, so Eq. [6.7] is used to calculate R_{bolts}.

Once again, the predictions of joint strength made using the recommendations of Section 6.2 are very close to the test values. On average, the ratio of calculated load to test load was 0.98, standard deviation 0.06. In all cases, the ultimate joint strength was governed by the combined action of the bolts and the welds.

6.5 Discussion of Results

The values of joint shear strength made using the design recommendations presented herein are in good agreement with the experimental values taken from three different sources of data. Each of the testing programs investigated had a different configuration of combination joint. The number of bolts used, for example, ranged from one to four. The length of transverse weld in a given specimen ranged from 355 mm to 520 mm and the length of longitudinal weld ranged from 279 mm to 560 mm. The good agreement found in the comparisons supports the notion that these design recommendations can be used for combination joints that are reasonably similar to those in the test programs described.

It appears that the design recommendations presented in Section 6.2 predict loads better for the experimental data of others than they do for the experimental data reported herein. This may seem unusual in light of the fact that these recommendations were developed using the experimental data presented herein. This can be explained, at least in part, by the following. The joints tested by Holtz and Kulak [1970] and Jarosch and Bowman [1985] were much less complicated than those tested as part of this study. Rather than using four bolts, for instance, only one or two bolts were used. Also, Holtz and Kulak did not do any tests combining both transverse and longitudinal welds with bolts. As the complexity of the joint increases, accuracy of the prediction decreases.

Moreover, twelve different types of combination joints were examined in this experimental study, whereas the studies conducted by Holtz and Kulak [1970] and by Jarosch and Bowman [1985] each had only three different types of combination joints. Given that the latter two testing programs had a smaller scope, it is understandable that the predicted values of load are closer on average.

Comparison between results obtained using the recommendations presented herein and rules given by the various codes and standards (Section 2.5) is difficult because the rationale used in each case is generally different. Using a hypothetical situation, some comparisons can be made, however. Consider, for example, an existing bolted joint that needs renovation as a result of new loading requirements. In order to be consistent with the tests reported herein, assume that the joint is connected using four 3/4 in. diameter bolts of ASTM grade A325 and that they are preloaded. The shear strength of these four bolts is 1396 kN, according the ancillary tests. Thus, assume that the original joint was designed to resist an ultimate load of, say, 1400 kN. Finally, assume that the additional loads being applied means that the renovated connection is required to resist a total force of 2000 kN.

Several possible designs for the renovated joint will be made, using the design rationale of the following three standards—the CSA S16.1 standard [1994], the AISC LRFD specification [1993], and the AISC ASD specification [1989]. In order to make the comparison of these design standards with the design recommendations of Section 6.2, all the predictions will use values for bolt and weld shear as found in the ancillary tests of this study (Table 4.3 and 4.6). Also, to simplify the comparisons, the design of the renovated joint will be restricted to the addition of either longitudinal or transverse fillet welds, but not both, with a nominal weld leg size of 6 mm. It is also necessary that an estimate be made concerning how much of the original load is considered dead load, and how much of it is live load. A reasonable live load to dead load ratio is 1.5. Thus, it will be taken that the original dead load was 560 kN, and the original live load was 840 kN. Because this joint is presumed to be in a loaded condition before the alteration, it will be assumed that the bolts are in a positive bearing condition.

For bolted joints that are to be strengthened by welding, both the S16.1 standard [CSA, 1994] and the LRFD standard [AISC, 1993] specify that the welds should be proportioned to carry all the load except the original dead load. That means that in this example, the bolts are to carry the original dead load of 560 kN and the welds must carry the remaining 1440 kN. According to these standards then (and using the weld shear strength found in the experimental study reported herein), a weld length of 690 mm is required in order for the joint to resist the 2000 kN force. In conformity with the recommendations made in Section 6.2, if the combination joint has this amount of weld length its total resistance is 2250 kN. Thus, for this example, the ratio of load calculated using the design standards S16.1 and LRFD to load calculated using the recommendations is 0.89.

If transverse welds instead of longitudinal welds are used in the hypothetical case described above, a weld length of 525 mm is require to achieve the resistance of 2000 kN. According to the recommendations herein, the resistance of this combination joint (four bolts and 525 mm of transverse weld) is now only 1560 kN. The corresponding ratio of load calculated using the design standards to load calculated using the recommendations is 1.28.

When welded alterations are made to bolted joints, the ASD specification [AISC, 1989] says that the existing bolts can carry all the load present at the time of alteration. In other words, for the hypothetical case, the bolts are assumed to carry 1400 kN, and the welds must be proportioned to carry the additional 600 kN. If longitudinal welds are used, then a total length of 290 mm must be added to the joint in order to provide a total joint resistance of 2000 kN. However, according to the recommendations herein, this design gives a resistance of only 1420 kN. For this case, the ratio of load calculated using the design standard to load calculated using the recommendations is 1.41. If transverse welds are used rather than longitudinal welds, a total length of 220 mm must be added to the joint. According to the recommendations herein, this design would give a resistance of only 720 kN. Now, the ratio of load from the design standard to load from recommendations is 2.78.

(In the foregoing example, it has been taken implicitly that conditions at ultimate load will be directly proportional to conditions at service loads. The ASD specification is an allowable stress standard, not an ultimate strength standard.)

For this postulated example, the load resistances calculated using the three design standards are very different from the load resistances calculated using the design recommendations of Section 6.2. The S16.1 standard [CSA, 1994] and the LRFD specification [AISC, 1993], for instance, give a relatively stronger design than the recommendations herein when longitudinal welds are used, and a relatively weaker design when transverse welds are used. The ASD standard [AISC, 1989] gives a much weaker design for both cases. The amount of live load and dead load present in a connection does not really affect the connection performance. The total load in the connection and the bearing condition of the bolts are the two most important factors to be considered. Furthermore, there must be a distinction between the case of load sharing of transverse welds and bolts and the case of load sharing of longitudinal welds and bolts. The three design standards do not make this distinction; thus, their designs made using transverse welds are all much stronger than the design recommendations of Section 6.2 predict. In practice, it is quite possible that these three design standards could produce very unconservative designs.

Table 6.1 Comparison of Recommendations with Test Results (Chapter 4)

Test No.	Weld Leg Size		Calculated Loads					R_{test} kN	Ratio of Calc. to Test
	Trans. Weld mm	Long Weld mm	R_{long} kN	R_{trans} kN	R_{frict} kN	R_{bolts} kN	$R_{ult\ joint}$ kN		
NSL-1	6.28	0	1224	0	0	0	1396*	1579	0.88
NSL-2	5.88	0	1146	0	0	0	1396*	1513	0.92
NPL-1	6.28	0	1224	0	115	0	1396*	1776	0.79
NPL-2	6.16	0	1200	0	115	0	1396*	1706	0.82
PSL-1	6.30	0	1228	0	0	1047	2275	2233	1.02
PSL-2	6.36	0	1239	0	0	1047	2286	2378	0.96
PPL-1	6.19	0	1206	0	115	1047	2368	2418	0.98
PPL-2	6.45	0	1257	0	115	1047	2419	2428	1.00
NST-1	0	6.37	0	1517	0	0	1517	1380	1.10
NST-2	0	6.29	0	1498	0	0	1498	1400	1.07
NPT-1	0	6.06	0	1443	115	0	1558	1676	0.93
NPT-2	0	6.05	0	1441	115	0	1556	1685	0.92
PST-1	0	6.08	0	1448	0	0	1448	1474	0.98
PST-2	0	6.51	0	1550	0	0	1550	1584	0.98
PPT-1	0	6.48	0	1543	115	0	1658	2111	0.79
PPT-2	0	6.67	0	1589	115	0	1704	1965	0.87
NSA-1	6.55	5.78	1256	1377	0	0	2445	2231	1.10
NSA-2	6.26	5.75	1200	1369	0	0	2389	2195	1.09
NPA-1	6.68	5.90	1280	1405	115	0	2608	2390	1.09
NPA-2	6.56	5.93	1258	1413	115	0	2597	2428	1.07
PSA-1	6.39	6.00	1226	1431	0	0	2473	2558	0.97
PSA-2	6.28	5.85	1205	1393	0	0	2417	2458	0.98
PPA-1	6.49	6.18	1246	1472	115	0	2646	2894	0.91
PPA-2	6.67	6.20	1280	1476	115	0	2679	2692	1.00

* Denotes cases where $R_{ult\ joint}$ is governed by the ultimate bolt shear capacity

Table 6.2 Comparison of Recommendations with Holtz and Kulak [1970]

Test No.	Leg Size mm	Weld Length mm	Weld Type	Calculated Loads				R_{test} kN	Ratio of Calc. to Test
				R_{weld} kN	R_{frict} kN	R_{bolts} kN	$R_{ult\ joint}$ kN		
BW-L-0-1	9.40	313	long.	797	0	330	1127	1170	0.96
BW-L-0-2	8.94	308	long.	745	0	330	1075	996	1.08
BW-L-0-3	8.43	304	long.	694	0	330	1024	952	1.08
BW-L-1-1	9.45	300	long.	769	54	330	1153	1117	1.03
BW-L-1-2	9.27	300	long.	753	54	330	1083	1090	0.99
BW-L-1-3	9.60	304	long.	790	54	330	1120	1174	0.95
BW-T-1-1	8.84	355	trans.	1234	27	0	851	1217	1.04
BW-T-1-2	9.32	363	trans.	1329	27	0	916	1334	1.00
BW-T-1-3	9.86	357	trans.	1381	27	0	952	1446	0.96

Table 6.3 Comparison of Recommendations with Jarosch and Bowman [1985]

Test No.	Calculated Loads					R_{test} kN	Ratio of Calc. to Test
	R_{trans} kN	R_{long} kN	R_{frict} kN	R_{bolts} kN	$R_{ult\ joint}$ kN		
WLB2-1	0	787	54	330	1171	1161	1.01
WLB2-2	0	787	54	330	1171	1094	1.07
WTB2-1	697	0	54	0	751	778	0.97
WTB2-2	697	0	54	0	751	812	0.93
LTB2-1	697	669	54	0	1420	1559	0.91
LTB2-2	697	669	54	0	1420	1446	0.98

7. SUMMARY AND CONCLUSIONS

7.1 Summary

The literature review identified that there has been very little research done on the behaviour of combination joints. The tests that have been done were on fairly small connections limited to only one or two bolts. The most notable experimental studies on combination joints were done by Holtz and Kulak [1970] and Jarosch and Bowman [1985].

In the research study reported herein, a total of 24 full-scale combination joints (containing high-strength bolts and fillet welds in the same shear plane) were tested in direct tension. A number of issues not previously investigated in detail by others were examined in this research program. The bearing condition of the bolts, for instance, is an issue that was explored thoroughly in this study. The location of the bolts relative to their holes is perhaps the most important factor in a combination joint in determining the amount of load sharing that exists between the individual fasteners. In one-half of the tests reported herein a condition of *negative bearing* was modeled; in the other half a condition of *positive bearing* was used. The effect of the bearing condition was quantified using a direct comparison between these two set of test results. A condition of positive bearing condition had been attempted by Holtz and Kulak [1970] by using "fitted" bolts, but it was done with limited success.

Another issue addressed in this study is the effect of plate friction in combination joints at ultimate load. In half of the tests reported herein, the high-strength bolts were tightened only to a snug-tight condition; in the other half of the tests, the bolts were preloaded using the turn-of-nut method. The effects of plate friction were quantified using a direct comparison between these two sets of results.

Based on their research, Holtz and Kulak [1970] recommended that the number of bolts used in a combination joint be limited because generally the bolts were not being loaded effectively. This notion was tested somewhat because the specimens of the research program reported herein all had four bolts. This is twice as many bolts as were used in the other two programs.

An ancillary testing program was also performed as a part of this research study. Results of these tests were used to develop load versus deformation curves for the individual fastening elements contained within the full-scale combination joints. Using the equations of load versus deformation developed from the ancillary tests and the analysis of the full-scale tests, estimates were made of the individual contributions to connection resistance from each of the individual fasteners. These estimates were subsequently used to develop a number of design recommendations for combination joints.

7.2 Recommendations

To calculate the total connection resistance of a combination joint based on the individual components of resistance $R_{friction}$, R_{bolts} , R_{trans} , and R_{long}, the following are recommended:

1. The connection resistance contributed by plate friction, R_{frict}, is calculated using Equation [6.1].
2. The connection resistance contributed by the transverse welds, R_{trans}, is calculated using Equation [6.2].
3. The connection resistance contributed by the longitudinal welds, R_{long}, is calculated using:
   a) Equation [6.3] for combination joints containing only longitudinal welds.
   b) Equation [6.4] for combination joints containing both longitudinal and transverse welds
4. The connection resistance contributed by bolt shear, R_{bolts}, is calculated using:

a) Equation [6.5] for combination joints with transverse welds and for combination joints with bolts not in a positive bearing condition.

b) Equation [6.6] for combination joints with only longitudinal welds and bolts in a positive bearing condition (experimental conditions).

c) Equation [6.7] for combination joints with only longitudinal welds and bolts in a positive bearing condition (field conditions).

In design, it is recommended that the ultimate strength of a combination joint, $R_{\text{ult joint}}$, be taken as the greatest of:

1. The ultimate strength of the bolts only.
2. The ultimate strength of the welds only.
3. The combined strength of the longitudinal welds and the high-strength bolts as calculated using Equation [5.5].
4. The combined strength of all the welds with the high-strength bolts, as calculated using Equation [5.5].

7.3 Conclusions

Perhaps the most important conclusion that can be drawn from this study is that transversely oriented fillet welds should not be used in combination joints. Even under the simulated condition of positive bearing (the so-called "best case"), the bolts in combination joints containing transverse welds contribute very little, if any, to the connection resistance through shearing action. This is an issue that is not addressed in any of the design standards examined in Chapter 2. Current design standards that allow this type of combination joint could possibly produce unconservative designs. Based on their research, both Holtz and Kulak [1970] and Jarosch and Bowman [1985] came to this same conclusion.

The design of combination joints should be based on the total load expected to be exerted on the connection. Furthermore, the strength of a joint should be evaluated based on the bearing condition, the size, and the quantity of the bolts, and the size and orientation of the welds. According to the S16.1 standard [CSA, 1994] and the LRFD standard [AISC, 1993], the strength of combination joints in which welds have been added to an existing bolted joint is evaluated based on the proportion of the dead and live load in the connection. The ASD standard [AISC, 1989] allows preloaded bolts to carry all the original load when welds are added to an existing connection. In some situations, these standards may produce unconservative designs.

In the two American specifications, LRFD [AISC, 1993] and ASD [AISC, 1989], combination joints containing bolts designed for bearing-type load transfer are not considered to share load with welds. According to these rules, the welds in this type of connection must be proportioned to carry the entire load. The S16.1 standard [CSA, 1994] does allow load sharing between welds and bolts designed for bearing-type load transfer, but only in renovations, not in new construction. Results of the tests reported herein indicate that bolts only tightened to a snug-tight condition can share load with welds. The bearing condition of the bolts at the start of loading, on the other hand, is a key factor in making this load sharing possible. In order to consider the amount of load sharing in this situation, there must be a reliable method of determining the actual bearing condition of the bolts. More research will be needed in this area in order to develop rules for safe design of combination joints.

The contribution of plate friction to the total ultimate resistance of the connection was not easily predictable, but it was always noticeable. Based on a relatively limited number of tests, a minimum value of plate friction contribution was assigned. This value should be verified with further experimentation.

REFERENCES

American Institute of Steel Construction, Inc., (1993) "Load and Resistance Factor Design Specification for Structural Steel Buildings," Chicago, Illinois.

American Institute of Steel Construction, Inc., (1989) "Allowable Stress Design and Plastic Design," Chicago, Illinois.

American Society for Testing and Materials, (1992) "A370-92, Standard Test Methods and Definitions for Mechanical Testing of Steel Products."

American Society for Testing and Materials, (1992) "A325-92a, Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]."

British Standards Institution, (1985) BS 5950: Part 1, "Structural use of Steelwork in building."

Butler, L.J., Pal S., and Kulak, G.L., (1972) "Eccentrically Loaded Welded Connections," Journal of the Structural Division, ASCE, Vol. 98, No. ST5, May, pp. 989 to 1005.

Canadian Standards Association, (1994) CAN/CSA-S16.1-M94, "Limit States Design of Steel Structures," Rexdale, Ontario.

Crawford, S.F. and Kulak, G.L., (1971) "Eccentrically Loaded Bolted Connections," Journal of the Structural Division, ASCE, Vol. 91, No. ST3, March, pp. 765 to 783.

European Committee for Standarisation, (1992) Eurocode 3 : Design of Steel Structures, ENV, 1993-1-1, Brussels.

Fisher, J.W., (1965) "Behavior of Fasteners and Plates with Holes," Journal of the Structural Division, ASCE, Vol. 91, No. ST6, Proc. Paper 4511, October, pp. 129 to 154.

Holtz, N.M. and Kulak, G.L., (1970) "High Strength Bolts and Welds in Load Sharing Systems," Department of Civil Engineering, Nova Scotia Technical College, Halifax, Nova Scotia.

International Organization for Standardization, (1994) ISO/DIS 10721, "Steel structures - Materials and design : Draft International Standard."

Jarosch, K.H., and Bowman, M.D., (1985) "Behavior of Tension Butt Joints Using Bolts and Welds in Combination," Structural Engineering Report CE-STR-85-20, Purdue University, June.

Kulak, G.L., Fisher, J.W., and Struik, J.H., (1987) Guide to Design Criteria for Bolted and Riveted Joints, Second Edition, John Wiley and Sons Inc.

Lesik, D.F., and Kennedy, D.J.L., (1988) "Ultimate Strength of Eccentrically Loaded Fillet Welded Connections," Structural Engineering Report No. 159, Department of Civil Engineering, University of Alberta, Edmonton.

Miazga,G.S., and Kennedy,D.J.L., (1986) "Behaviour of Fillet Welds as a Function of the Angle of Loading," Structural Engineering Report 133, Department of Civil Engineering, University of Alberta, Edmonton.

Swiss Society of Engineers and Architects, (1979) SN 555 161, "Sia : Steel Structures," Standard 1979 Edition.

Wallaert, J.J., and Fisher, J.W., (1965) "Shear Strength of High-Strength Bolts," Journal of the Structural Division, ASCE, Vol. 91, No. ST3, June, pp. 99 to 125.